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Survival of Reinforced Concrete Flat Plate System against Column Loss

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ABSTRACT: Column loss in a flat plate building, due to punching shear, explosion, impact load or any accidental event, can lead to what is termed progressive collapse. Progressive collapse is inherently a dynamic process, which makes it difficult to experimentally explore structures with real scale. Therefore, this paper aims to numerically investigate the behavior of flat plate systems due to column loss utilizing nonlinear finite element analysis with the aid of the computer software (ABAQUS). In this investigation, the nonlinear dynamic response of both an old flat plate building, designed according to the ACI 318-71, and a similar modern building, designed according the ACI 318-14, subjected to an instant removal of a column is examined. The obtained results clearly reveal that the old flat plate building without continuous slab bottom reinforcement at columns is highly vulnerable to progressive collapse and the efficiency of the continuous bottom reinforcement within the column strip, as recommended by the ACI 318-14, in preventing the disproportion or progressive collapse of a reinforced concrete flat plate building. Such reinforcement is able to produce alternate load path through the tensile membrane action, thus providing ductility and robustness in the system.

KEYWORDS: progressive collapse, flat plate, column loss, nonlinear finite element analysis, membrane action, robustness

1 INTRODUCTION

A reinforced concrete flat plate construction system is being widely used in residential and industrial buildings in many parts of the world since it reduces both the formwork cost and construction time and eases installation. In addition, the reduced story height and the increased architectural freedom in design minimize the overall construction and maintenance costs. However, in this system, the slab-column connection is prone to a brittle punching shear failure under gravity and/or earthquake loads or any abnormal events, which may cause progressive or disproportionate-collapse.

The terms progressive- and disproportionate-collapse have been found in many technical papers, with different definitions for both, making it difficult to understand the clear difference between them. Progressive-collapse is defined as the spread of an initial local failure from element to another resulting in the collapse of the entire structure or a disproportionately large part of it (ASCE, 2010). Disproportionate-collapse is defined as a collapse which results from small damage or a minor action leading to the collapse of a relatively large part of the structure, such as the Ronan Point building collapse where a small portion of the overall structure failed (Fu, 2016). Causes of progressive collapse can be attributed to gas or bomb explosions, fire, collisions of vehicles or airplanes, wind tornadoes, faulty construction, foundation failure, construction failures, design errors, or other extreme events. In other words, progressive-collapse can be caused by any abnormal event that leads to the failure of a key structural element. There are several famous examples of progressive collapse such as the collapse of the Twin Towers on September 11, 2001, due to aircraft impact and the collapse of World Trade Center 7 later, same day, due to a fire set by the debris of the Twin Towers (Fu, 2016). The partial collapse of the Ronan Point building was triggered by an internal gas explosion in London in 1968 whereas the partial collapse of the Alfred P. Murrah Federal building in 1995 was induced by a blast (Fu, 2016). For space structures, there is the famous collapse incident at the Paris airport (Fu, 2016). The space frame of the Hartford Civic Center in the United States collapsed in 1978 due to heavy snow (Fu, 2016). Bridge collapse, as another quite common type of progressive collapse, can be attributed to impact loading from the collision of a ship or from overloaded trucks. A recent example is the progressive collapse of the suspension bridge Kutai Kartanegara in East Borneo, Indonesia (Fu, 2016).

According to the Department of Defense (DoD, 2009) and General Service Administration (GSA, 2003), one of the currently used approaches to assess the robustness of a structure against progressive collapse is the *alternate load path method*. In approach, the response of a structure subjected to an instant removal of one or more load-bearing components is to be examined.

Compared with modern construction, older flat plate structures in the United States without continuous slab bottom reinforcement at columns (before the ACI 318-89) did not inherit structural integrity after a punching shear failure is started at a slab-column connection. This category of flat plate buildings is thus deemed more vulnerable to progressive collapse. Generally, as per the ACI Code (ACI 318-14), part of the longitudinal bottom reinforcement in the column strip is required to continue through the column in order to prevent progressive collapse. Tensile membrane action in a flat plate system may become the main load redistribution mechanism after a column loss.

This paper presents a numerical study via the finite element using ABAQUS software (ABAQUS, 2013) in order to evaluate the nonlinear dynamic responses of both an older (ACI 318-71) and a modern (ACI 318-14) four bays-four stories reinforced concrete flat plate buildings subjected to an instant removal of column. Each building is analyzed for gravity loads with the simulation of the loss of (1) an interior column, (2) an exterior column, and (3) a corner column, each at a time.

2 ANALYSIS

2.1 *Finite Element Model*

In the three-dimensional nonlinear dynamic finite element analysis utilized in this study, the software ABAQUS (ABAQUS, 2013) is used, Figure 1. Eight-node continuum 3D solid shell element C3D8R is used to model concrete in the slab and column elements. As for reinforcement it can be defined as either discrete elements or distributed in a finite element model of reinforced concrete components. In the discrete approach, the reinforcing bars are explicitly modeled using truss elements compatible with the surrounding elements of concrete. This approach is computationally demanding for a system-level analysis. As for the distributed approach, reinforcement is defined as a rebar layer based on the cross-sectional area of each rebar, spacing, location, and orientation with respect to a local coordinate system. The discrete approach is adopted in the current study. The embedded technique is also used to model the connectivity between concrete and reinforcement nodes in order to simulate perfect bond between the two materials.

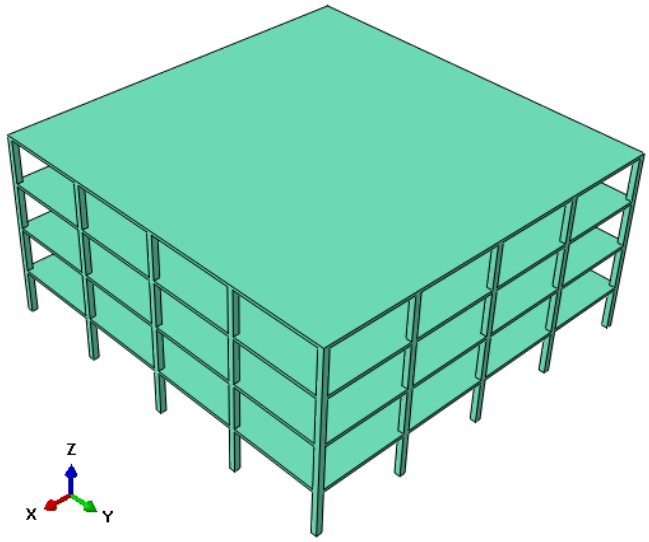


Figure 1. 3D finite element model based on ABAQUS package (ABAQUS, 2013).

2.2 *Materials*

In the analysis, the well-known and widely accepted model suggested by (Hognestad, 1951) is employed to model the uniaxial compressive behavior of concrete. This model, together with that for concrete in tension (ABAQUS, 2013) is shown in Figure 2. The term "Concrete damage plasticity model" in ABAQUS is adopted to define the concrete constitutive relationship under a triaxial state of stress. This model is suitable for rate-sensitive analysis, such as dynamic progressive collapse analysis, and can be applied to both implicit and explicit algorithms (ABAQUS, 2013). Since the latter is a powerful tool to overcome divergence problem in both dynamic and progressive collapse analyses, it is adopted in this study.

Utilizing the concrete damage plasticity model, other property parameters (such as the dilation angle, etc.) for concrete biaxial and triaxial behavior from classical tests are recommended. The dilation angle is considered as 35 degrees for low to moderate confined concrete (Mercan et al., 2010). In addition, to define the yield surface, the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress is considered 1.16. Also, a value of 0.667 for the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian is used. The Poisson's ratio of concrete is assumed as 0.2.

The material properties of all slab reinforcement were introduced using an elastic-perfectly plastic material model. The parameter Nlgeom (geometric nonlinearity setting) = [on] in the step command can be used to perform the nonlinear geometric analysis in ABAQUS.

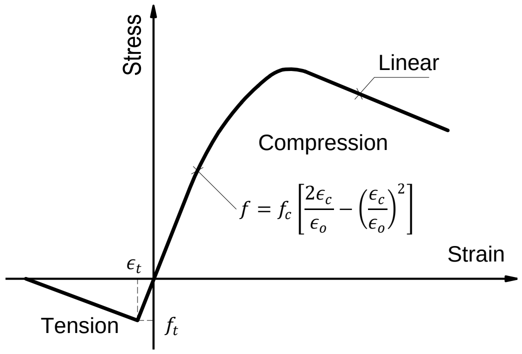


Figure 2. Uniaxial stress-strain relationships of concrete (Hognestad, 1951 and ABAQUS, 2013).

3 SIMULATION PROCEDURE OF COLUMN LOSS

3.1 *Approach*

The loss of a column due to an explosion, for instance, is a dynamic process; therefore, the sudden failure of a column can be modeled by replacing the potential failed column in Figure 3a with its reaction *R*, Figure 3b, and an equivalent opposite point load at the top of the column position, Figure 3c. At the first step, the gravity load and column reaction, *W* and *R*, are applied gradually, Figure 4. The second step is conducted by keeping the gravity load and column reaction for a certain time and then the column reaction is released suddenly as shown in Figure 4 by adding the downward load *R* at the top of the column position. In this study the gravity loads accompanied with the equivalent column reaction are ramped to their full amount during 0.5 second and kept unchanged for another 0.5 second to avoid dynamic effect at this stage, Figure 4. Then, the column reaction is suddenly removed with extending the time history of the gravity loads for additional 2.0 seconds.

3.2 *Static Analysis*

Following the DoD guideline (2009), a service gravity load *W* consisting of 1.20 times the dead load plus 0.50 times the live load is statically applied to the floor slabs. The point loads at the top of the interior column C3, the exterior column C5, and the corner column A5 (to be removed at the first story) are 1123.1kN, 549.5kN, and 275.1kN, respectively. The interior, exterior, and corner columns are then deleted, each a time, from the original model and the previously determined column loads are applied as opposite reactions at the junction between the slab and the removed column, Figure 3b. Thus, replacing a column with its reaction simulates the intact structure (original structure).

3.3 *Dynamic Analysis*

As shown in Figure 3c, the sudden loss of a column (e.g., C3, C5 or A5) is modeled by instantly applying a downward load to cancel the column reaction (determined from static analysis) at the slab-column joint. The DoD guideline (2009) requires that, when nonlinear dynamic analysis procedure is employed, the duration for column removal shall be less than 0.10 times the natural period associated with the vertical vibration of the bays above the removed column; the natural period of the presented case study is 0.335 second. In addition, from the field test of a 10-story concrete frame structure by Sasani et al. (2007), the time spent on column removal by explosion was about 0.005 second. Therefore, the reaction force was applied dynamically within about 0.005 second, Fig. 4. As already stated in previous, the dynamic explicit solver is adopted because of its advantage in solving problems associated with large deformation and extremely discontinuous events (ABAQUS, 2013).

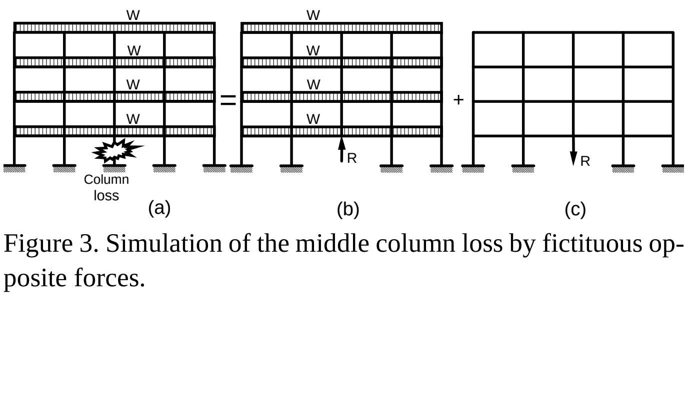


Figure 3. Simulation of the middle column loss by fictituous opposite forces.

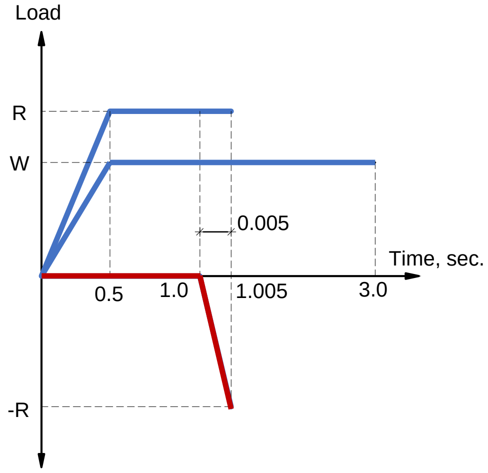


Figure 4. Load- time curves for dynamic progressive collapse analysis procedure.

4 CASE STUDY I - OLDER FLAT PLATE BUILDING (ACI 318-71)

4.1 Building Description

Figure 5 shows a four-story flat plate building with office occupancy. It is designed according to the ACI 318-71 and has a story height of 3050mm and an equal span length of 6096mm in each direction. All the columns are square, 380 × 380mm. All floor slabs have the same thickness of 191mm. The columns to be individually removed in the analyses are the interior column C3, the exterior column C5, and the corner column A5, each at a time. The columns are all at the first story.

The service loads acting on the floor slabs include a live load of 2.40kN/m^2 and a dead load of 5.44kN/m^2 considering both the slab self-weight and the superimposed dead load due to partitions and floor finishing. It is assumed that the gravity load combination of 1.40DL+1.70LL controls the design. Grade 60 reinforcement with a yield stress $f_y =$ 414MPa and a concrete with a cylinder strength $f_c' = 27.6$MPa are assumed for all structural components. The clear cover of slab reinforcement is specified as 20mm.

Figures 5c and 5d show the slab reinforcement layout, bars of diameter 12.7mm, in one direction of the column and middle strips. The slab top reinforcement within the column strip is 0.60% in interior and exterior panels. As for the bottom reinforcement ratio it is 0.32% in the column strip of the exterior panels (end spans). Elsewhere, the top and bottom reinforcement is 0.21%, which is governed by code minimum requirements for controlling shrinkage and temperature effects. The ACI 318-71 permits the slab top bars having alternating lengths. However, to simplify the finite element modeling, the top bars are curtailed at one location on the basis of the average required length, Figure 5c. From the slab bottom reinforcement, only two bars are extended (not continued) into the columns for a length 114mm, and the rest of these bars are alternately curtailed, Figure 5d.

4.2 Building Response

4.2.1 Load-Deflection Curve at Removed Column

The sudden removal of a ground floor column causes significant downward deflection of the floor slabs at the location of the failed column. In this study case, the column was totally removed at time t = 1.005 seconds within a period of 0.005 second, Figure 4. Figures 6a, 7a and 8a show the deformed shape of the building models following the removal of the ground floor interior column C3, exterior column C5, and corner column A5, respectively, Figure 5. Figures 6b, 7b and 8b show the time history of the downward deflections of the points in the first floor above the considered failed columns, which have reached large values in all cases upon column loss. The figures show also the time at which the solution stopped due to excessive rotation of the slabs as a result of the fracture of the top bars at the columns neighboring the lost column. As per Figures 6b and 6e, punching failure happened at columns C4, C2, D3 and B3 when the deflection at the removed interior column C3 reached an approximately 82mm at time 1.22 seconds. However, these columns were still able to sustain loads through post-punching, Figure 6c. As per Figures 7b and 7e, punching failure happened at columns C4, D5, and B5 when the deflection at the removed exterior column C5 reached an approximately 100mm at time 1.22 seconds. However, as in the case of interior column loss, these columns were still able to sustain loads through post-punching, Figure 7c. As per Figures 8b and 8e, punching failure happened at columns A4 and B5 when the deflection at the removed corner column A5 reached an approximately 120mm at time 1.22 seconds. However, similar to the case of interior column loss, these columns were still able to sustain loads through post-punching, Figure 8c. The observed deformation characteristics of the different floor slabs were almost identical.

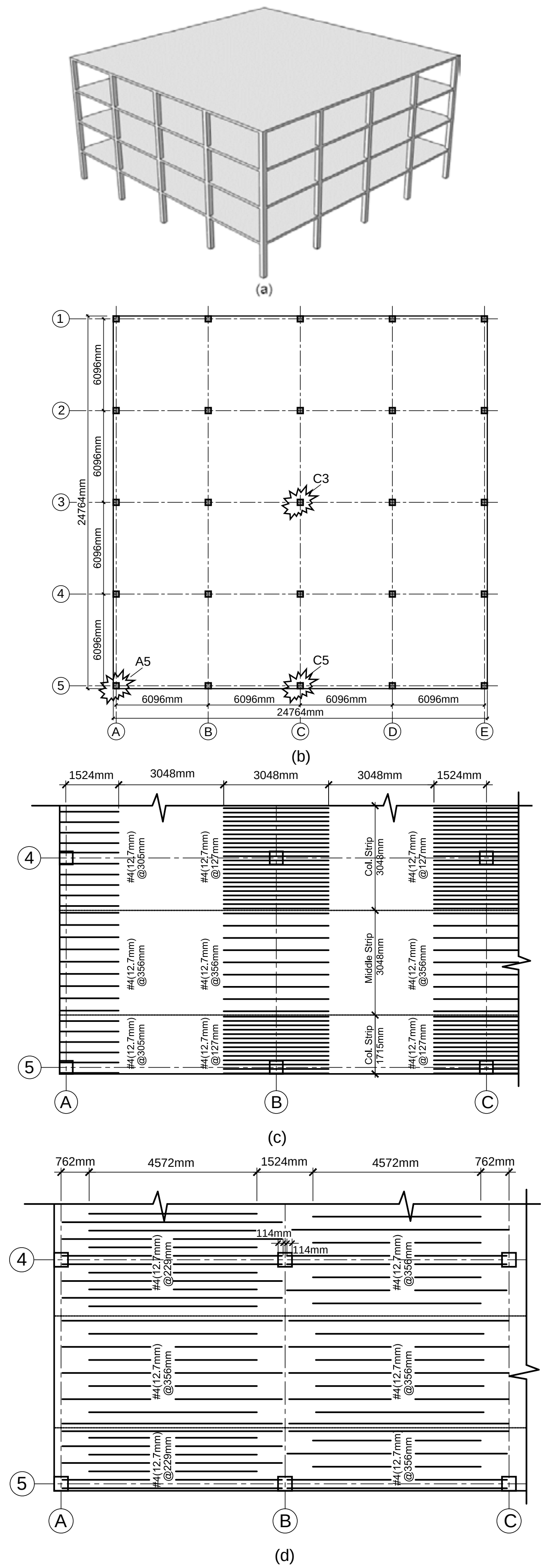


Figure 5. Flat plate building: (a) 3D view; (b) floor plan; (c) slab top reinforcement; and (d) slab bottom reinforcement.

4.2.2 Load Redistribution of Adjacent Columns

Upon the failure of any of the selected columns, its load is redistributed to the neighboring columns. Columns C4 (and C2) and columns B3 (and D3) in Fig. 6c, columns B5 (and D5) and column C4 in Fig. 7c, and columns B5 and A4 in Fig. 8c carried the majority of the redistributed loads as they are nearest to the removed columns in the three cases of column loss. Thus, they experienced overloading and subsequent punching failure in the connections, leading to progressive collapse of the entire system. Following the punching failures and excessive rotation of slab at the neighboring columns the solution has stopped.

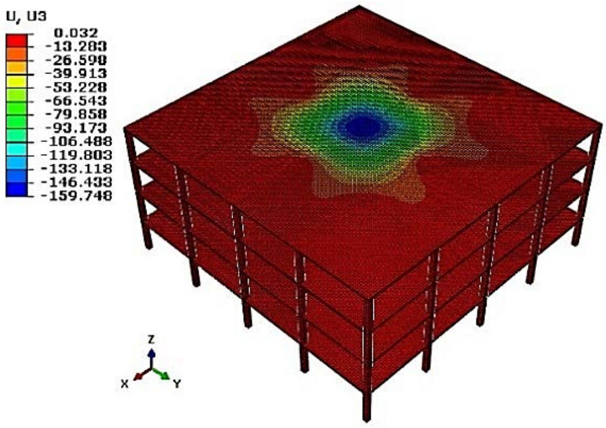
4.2.3 Slab Post-Punching

From literature (Peng, 2015), the post punching capacity of older flat plates without integrity bars consists of (1) breaking out of the concrete cover; and (2) the top bars are pulled out of concrete then fracture. This was observed in ABAQUS finite element model in the form of excessive rotation of slab due to fracture of top bars at the neighboring columns of the lost column.

Once a punching failure occurs, there is a transition from dowel action to tensile membrane action for resisting the load applied to a slab-column connection. Both actions depend on slab bottom bars because, upon punching failure, the top bars are stripped out of slab due to the spalling of slab concrete cover. For older flat plate buildings, the slab bottom bars are not adequately anchored into the column without sufficient development length and will be pulled out of column. Therefore, following the punching failure at the slab-column connection, the slab bottom bars give a limited residual strength due to dowel action and cannot be engaged to effectively develop a tensile membrane action but a limited residual strength has quickly started degrading and dropped down to zero due to the lack of continuity of slab bottom bars at the columns. This was observed in ABAQUS finite element model in all cases of column loss, the solution has stopped due to excessive rotation and fracture of slab top bars leading to punching failure (see Figures 6d and 6e) and then a progressive collapse of the entire system. Such behavior was observed in all floor slabs that cannot develop sufficient tensile membrane action in order to transfer gravity loads.

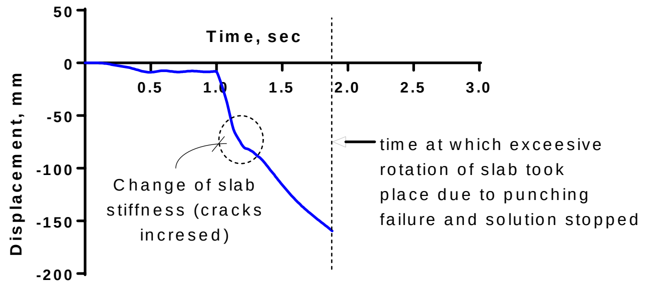
4.2.4 *Concrete Damage*

Damage is generally defined as the condition of the structure when it is not operating in its ideal condition but is still serviceable. In most cases, the damage in concrete starts with the initiation of cracks, which propagate and finally lead to collapse. Figures 6e, 7e and 8e show the tensile damage (crack pattern) of all cases at failure.

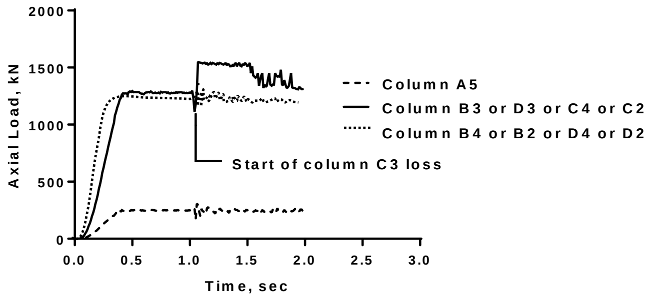


(a) Contours of vertical displacement

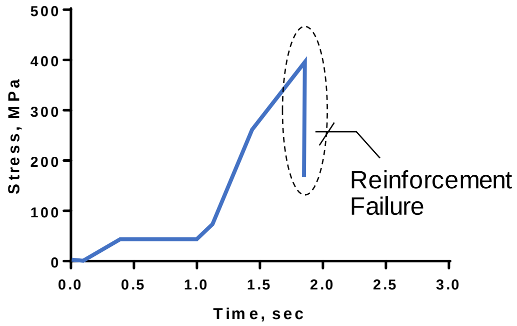
Figure 6. Case study I - the results upon the loss of the ground floor interior column C3.



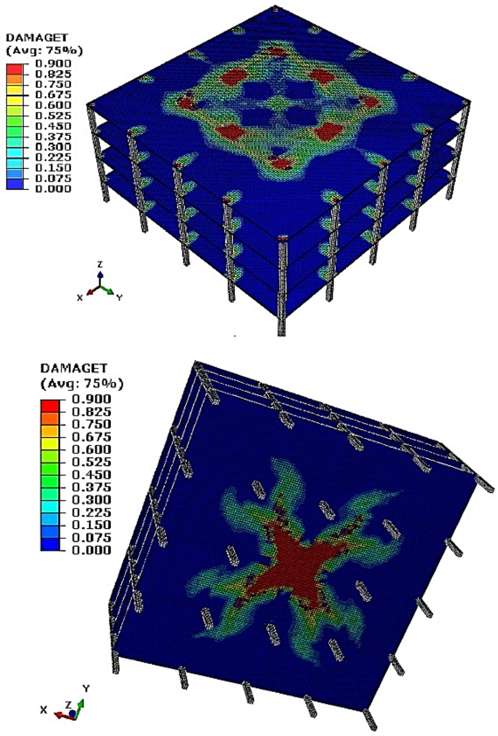
(b) The vertical deflection in the 1st floor above the lost column



(c) Load redistribution due to the loss of the interior column C3

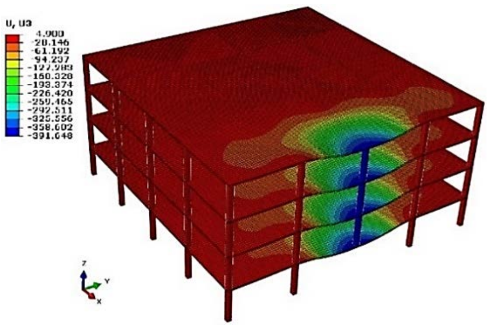


(d) Fracture of some top rebar



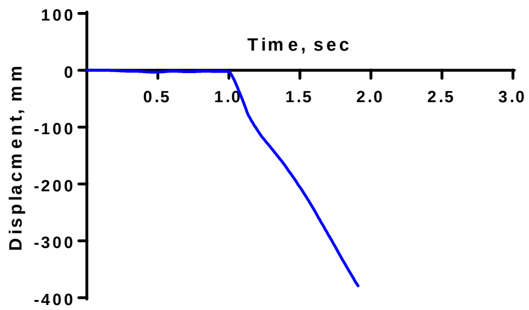
(e) Crack pattern near failure

Figure 6. Cont.

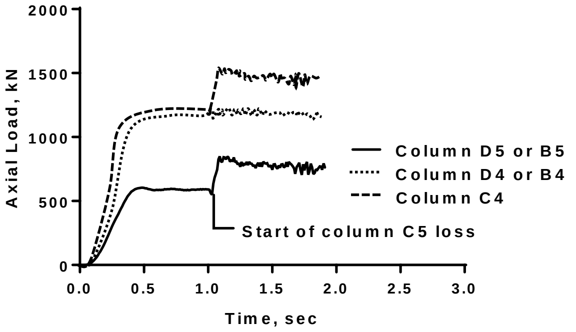


(a) Contours of vertical displacement

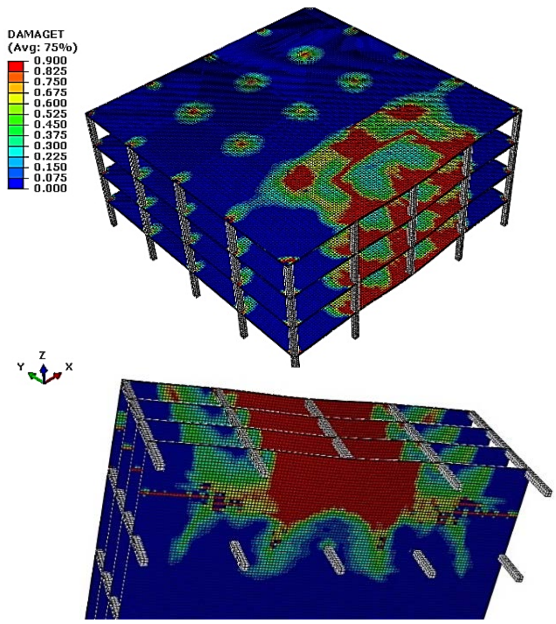
Figure 7. Case study I - the results upon the loss of the ground floor exterior column C5.



(b) The vertical deflection in the 1st floor above the lost column

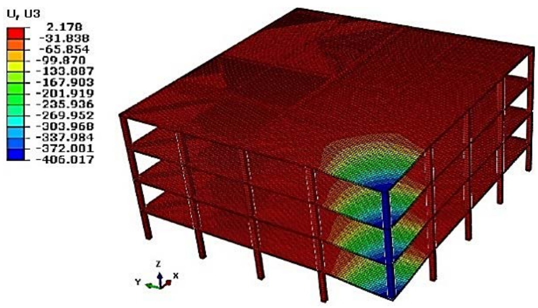


(c) Load redistribution due to the loss of the exterior column C5



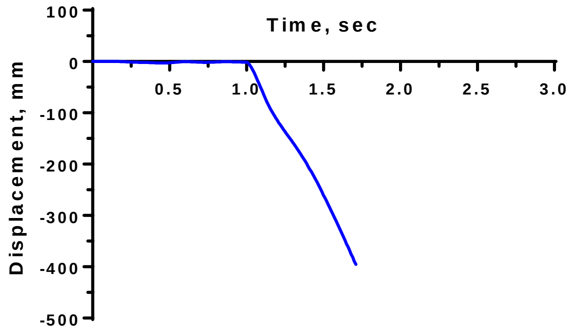
(d) Crack pattern near failure

Figure 7. Cont.

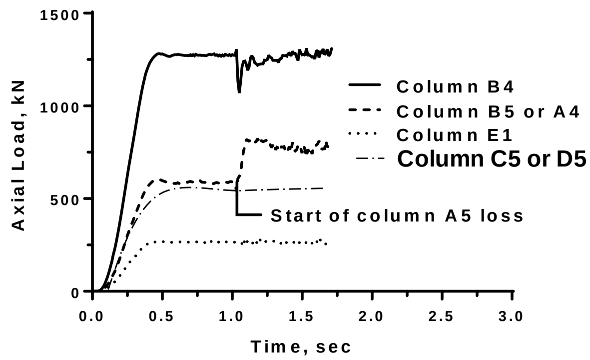


(a) Contours of vertical displacement

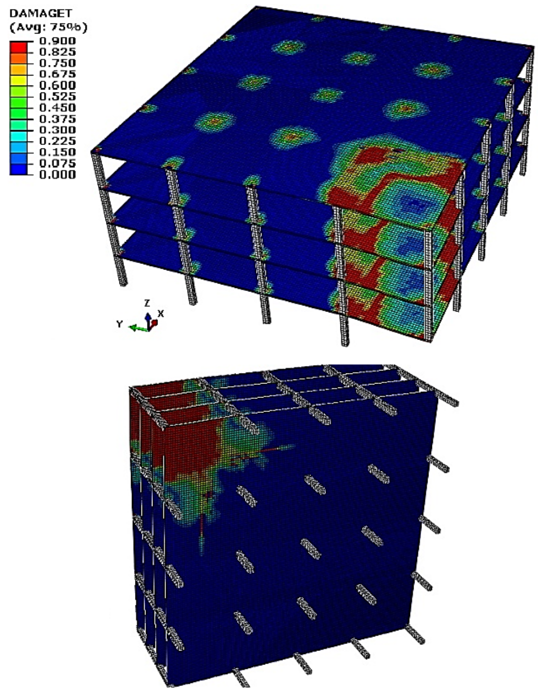
Figure 8. Case study I - the results upon the loss of the ground floor corner column A5.



(b) The vertical deflection in the 1st floor above the lost column



(c) Load redistribution due to the loss of the corner Column A5



(d) Crack pattern near failure

Figure 8. Cont.

5 CASE STUDY II - MODERN FLAT PLATE BUILDING (ACI 318-14)

5.1 Building Description

In this case, the building is similar in every aspect to the previous old building case except that a retrofitting scheme is followed in order to prevent progressive collapse. The scheme utilizes the reinforcement details of the modern ACI 318-14, Figure 9, where all bottom bars within the column strip, in each direction, are continued, Figure 10a. Arrangement of all bottom bars in each direction are shown in Figure 10b. This building has been analyzed and its performance is discussed in the following section.

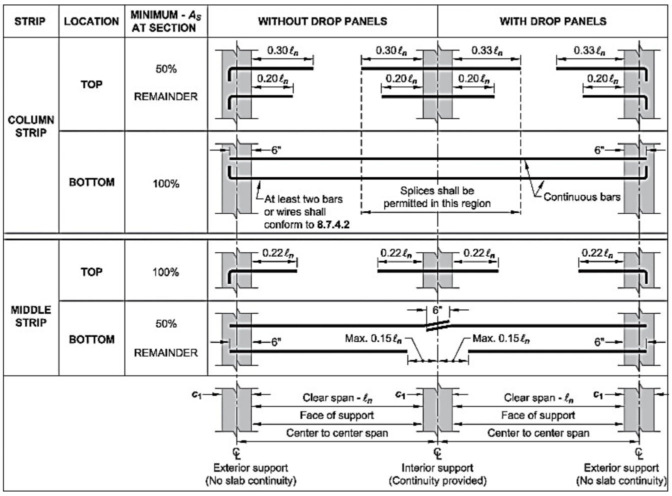
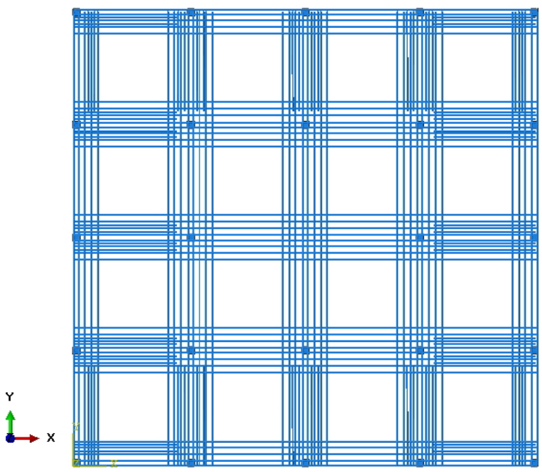
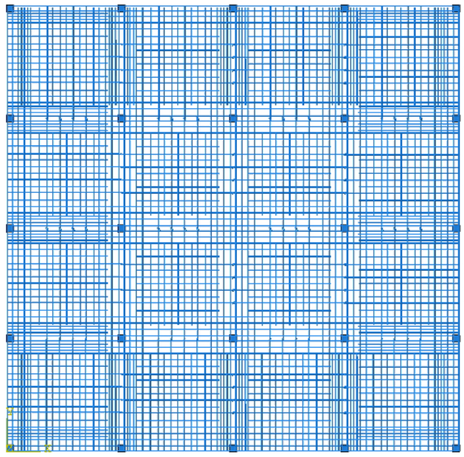


Figure 9. Reinforcement details of a modern flat plate building according to the ACI 318-14.



(a) Continuous bottom reinforcement within column strip



(b) Overall bottom reinforcement

Figure 10. Details of reinforcement in ABAQUS based on the ACI 318-14.

5.2 Building Response

5.2.1 Load-Deflection Curve at Removed Column

Removing a ground floor column suddenly causes large downward deflections at the points in the floors above the failed column. As per Figure 4, such a column was totally removed at time t = 1.005 seconds within a period of 0.005 second. The downward deflections of the positions of the lost columns in the first floor along their time history are shown in Figures 11a, 12a and 13a for the cases of interior column C3, exterior column C5, and corner column A5, respectively. In all cases of column loss, these deflections have reached abruptly peak values as a result of the sudden loss of the column and then remained in a steady state. Figure 11d show that the stress in the continuous bottom bars has reached yield. Nevertheless, as per Figure 11a, no progressive collapse has taken place in the building due to column loss. Similar behavior happened with removing the exterior column C5 and the corner column A5, Figures12 and 13, respectively.

5.2.2 Load Redistribution and Vierendeel or Frame Action

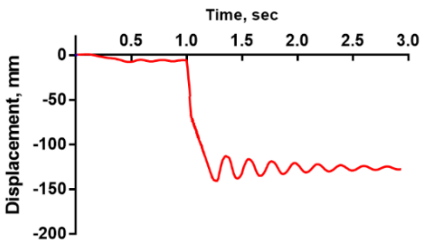
The vierendeel action is the primary mechanism for load redistribution, which makes all slabs and columns to work as one unit. Thus, it plays an important role in limiting the deformation of a system in comparison with the deformation of one slab alone, in the case of column loss. This behavior has been observed in this case study, Figures 11a, 12a and 13a.

When failure occurs at the interior column C3, its load is redistributed to the neighboring columns C4, C2, D3, and B3, Figure 11b. These columns, Figure 11b, carry the majority of the redistributed load as they are nearest to the removed column C3 and, thus, experience overloading. As shown in Figure 11a, no progressive collapse of the building happened. Similar response took place when removing the exterior column C5 or the corner column A5, Figures12 and 13, respectively.

5.2.3 Tensile Membrane Action

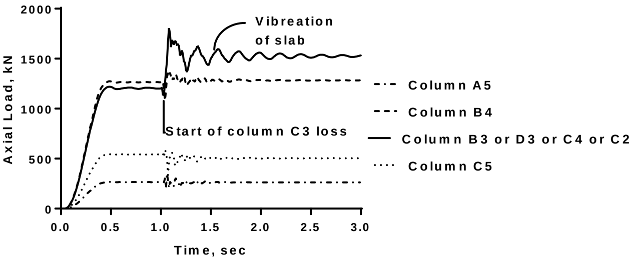
The continuous bottom reinforcement in column strip provides the slab some residual ability to span to the adjacent supports once a single support has been damaged. The continuous bottom bars in a column strip may be termed *integrity reinforcement* (ACI 318-14) and providing such reinforcement detailing will give the slab some residual strength upon a column loss by tensile membrane action, Figures 11f, 12b, and 13b. This concept is consistent with Mitchell and Cook findings (Mitchell and Cook, 1984) and the ACI 318-14 recommendation of adding continuous bottom bars.

The phase of tensile membrane action is preceded by a compressive membrane action. The latter action can be considered as a secondary load-resisting mechanism that resists progressive collapse just after a column loss, Figure 11f. Following the punching failures, the floor slab develops sufficient tensile membrane action to carry gravity loads, Figures 11g, 12b, and 13b. The presence of such robust behavior which is secured by placing continuous bottom reinforcement over supports, prevents the progressive collapse of the entire building.

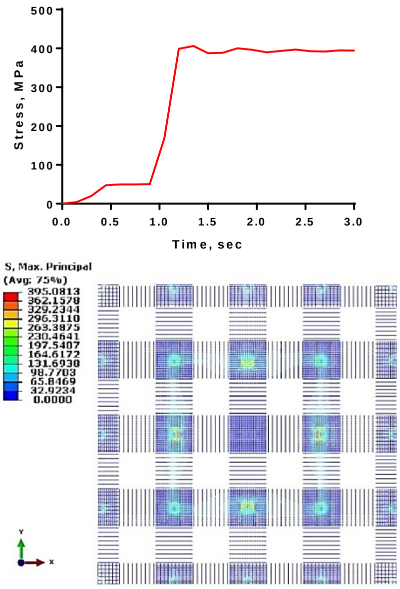


(a) The vertical deflection in the 1st floor above the lost column

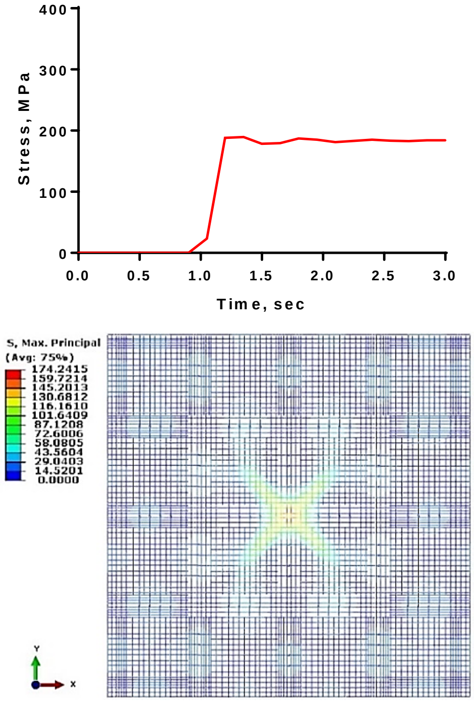
Figure 11. Case study II- the results upon the loss of the ground floor interior column C3.



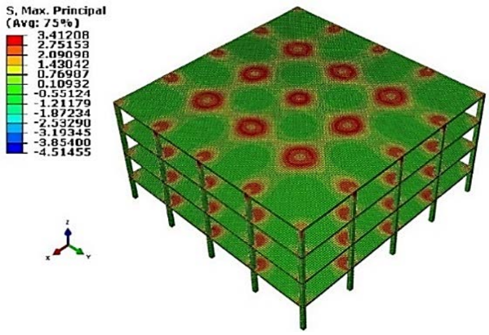
(b) Load redistribution due to loss of the interior column C3



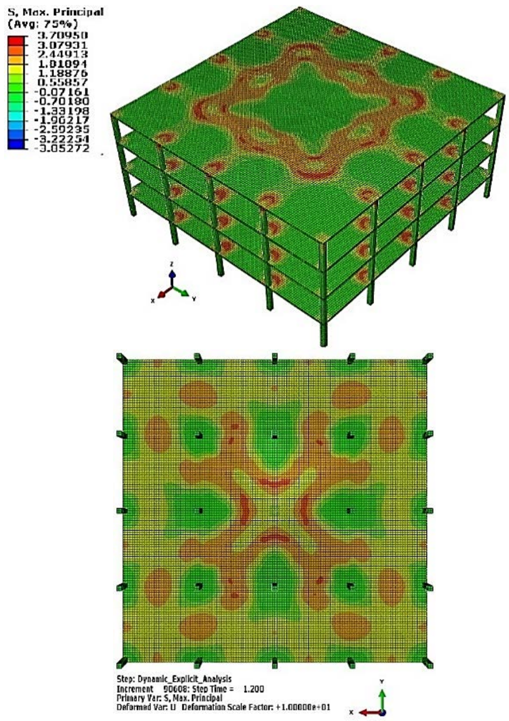
(c) Stresses in top bars within the column strip



(d) Stresses in continuous bottom bars

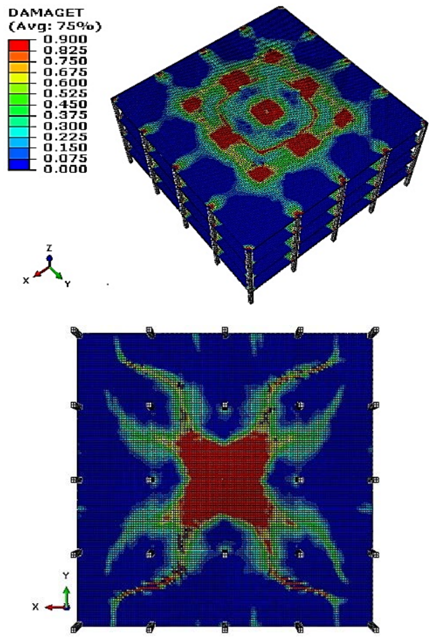


(e) Concrete stresses before the loss of interior column C3.



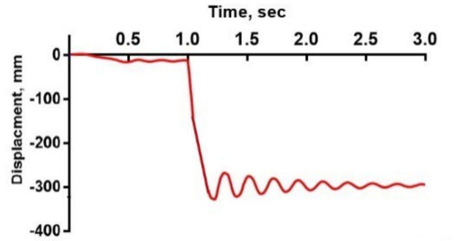
(f) Concrete stresses upon the loss of interior column C3

Figure 11 Cont.

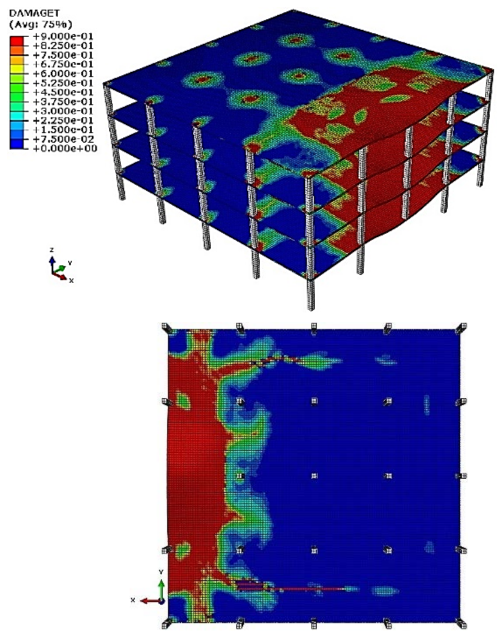


(g) Tensile damage (crack pattern)

Figure 11 Cont.

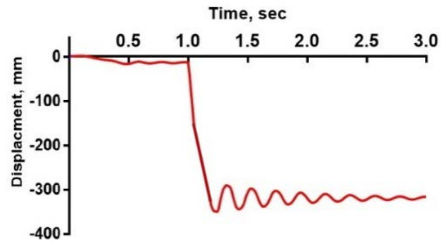


(a) The vertical deflection in the 1st floor above the lost column

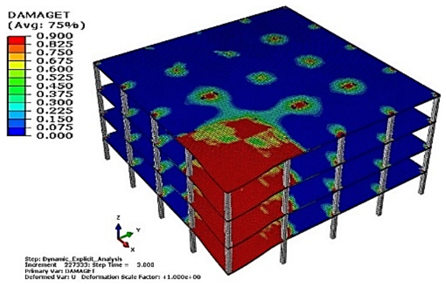


(b) Tensile damage (crack pattern)

Figure 12. Case study II- the results upon the loss of the ground floor exterior column C5.



(a) The vertical deflection in the 1st floor above the lost column



(b) Tensile damage (crack pattern)

Figure 13. Case study II- the results upon the loss of the ground floor corner column A5.

6. CONCLUSIONS

A nonlinear dynamic analysis of two flat plate buildings, one old and the other is modern, subjected to an instant loss of interior, exterior and corner column, each at a time, has been presented. The old building is designed based on the ACI 318-71 whereas the modern building confirms with the ACI 318-14. The analysis has been carried out with the aid of the finite element software ABAQUS, in which both material and geometric nonlinearities have been accounted for. From the obtained results, the following conclusions can be drawn:

1. The failure mode of an older flat plate building without continuous slab bottom reinforcement (upon the loss of interior, exterior or corner column, each at a time) consisted of punching shear failure accompanied by rupture of the slab reinforcement. As the floor slab cannot develop sufficient tensile membrane action to carry gravity loads, the entire building is prone to experience progressive collapse. Although the flat plate structure has exhibited adequate deformability, its resistance to progressive collapse is enhanced by ensuring the continuity of all slab bottom bars within the column strips.

2. Following the punching shear failures, the floor slabs develop sufficient tensile membrane action (an alternate load path) to carry gravity loads by placing continuous bottom reinforcement over supports within the column strips as proposed by the ACI 318-14, reflecting ductile and robust behavior, which enables the structural system to avoid disproportionate or progressive collapse.

3. Membrane action is the main mechanism for the load transfer of flat plate structures subjected to a ground column loss.

4. Losing an exterior or a corner column is particularly significant where the potential of progressive collapse in a reinforced concrete flat plate system is relatively large.

7. REFERENCES

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