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Assessment of progressive collapse potential in corner floor panels of reinforced concrete buildings

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ABSTRACT

The Department of Defense (DoD) document, UFC 4-023-23, which provides technical guidance for mitigation of progressive collapse, classifies buildings based on the desired level of protection. Medium and high levels of protection categories require the use of the Alternate Path (AP) method to investigate the capability of the structural system to transfer loads safely from a notionally removed column to the remaining structural elements. Certain columns and structural elements at prescribed locations must be investigated to determine the structural bridging capabilities over the removed column. Transfer of loads from a notionally removed corner column to the adjacent structural elements can impose significant stress/deformation demand on structural elements supporting the corner panel. When the panel area exceeds the floor damage limits, the panel and its structural elements must be designed to support the additional load or the loads must be transferred to adjacent columns. This paper investigates the implementation of UFC 4-023-23 to protect against progressive collapse of corner floor panels when their dimensions exceed the damage limits. A case study of a reinforced concrete building is analyzed, designed, and investigated using the AP method.

1. Introduction

Although UFC4-023-03 [2] is intended for the design of DoD facilities, it is the design document with the most detailed requirements for protection against progressive collapse and may provide a foundation for provisions that can be incorporated into general building codes such as the International Building Code (IBC). UFC limits the need for design against progressive collapse to buildings of three stories or more. The mitigation strategies discussed in this paper are for progressive collapse caused by the loss or damage of a structural element regardless of the cause. The strategies described in this paper are based on UFC and do not address strengthening of specific elements to prevent damage. A literature review on progressive collapse mitigation in codes and standard can be found is available [3].

Two levels of protection against progressive collapse are discussed in the UFC: (a) catenary action provided to the structure by tie forces in vertical and horizontal elements, and (b) flexural bridging capabilities that allow the structure to support loads transferred to adjacent elements from a notionally removed element using the AP method. The catenary action tie force requirements can generally be satisfied with limited calculations and good connection details.

Structures classified in the category of Very Low Level of Protection (VLLOP) or Low Level of Protection (LLOP) are protected against progressive collapse by providing adequate tie forces to develop catenary action in case of loss of structural element. If the specified vertical tie force cannot be provided, the AP method is used to ensure the structure is capable of bridging over a damaged element.

Structures classified in the category of High Level of Protection (HLOP) or Medium Level of Protection (MLOP) must have adequate tie forces in the vertical and horizontal directions, in addition to bridging capabilities over certain vertical elements. Predefined locations are specified for notional removal of columns to check for bridging capabilities. Fig. 1 shows the locations of external columns that are to be removed notionally to conduct a UFC AP investigation of a framed structure as well as the corner column that will be removed for the purposes of this study. UFC requires a separate AP investigation for each designated column at each floor level. Each analysis must demonstrate that the structure has the bridging capabilities after the notional removal of each selected column. If the structure cannot provide the bridging capabilities, elements must be redesigned. For large structures, this requirement can be very time-consuming.

A number of researchers has studied two-dimensional and three-dimensional modeling of structures for AP investigation of progressive collapse potential. Powel [4] concluded that static analysis is more conservative and that dynamic analysis is more accurate and no more difficult than static analysis.

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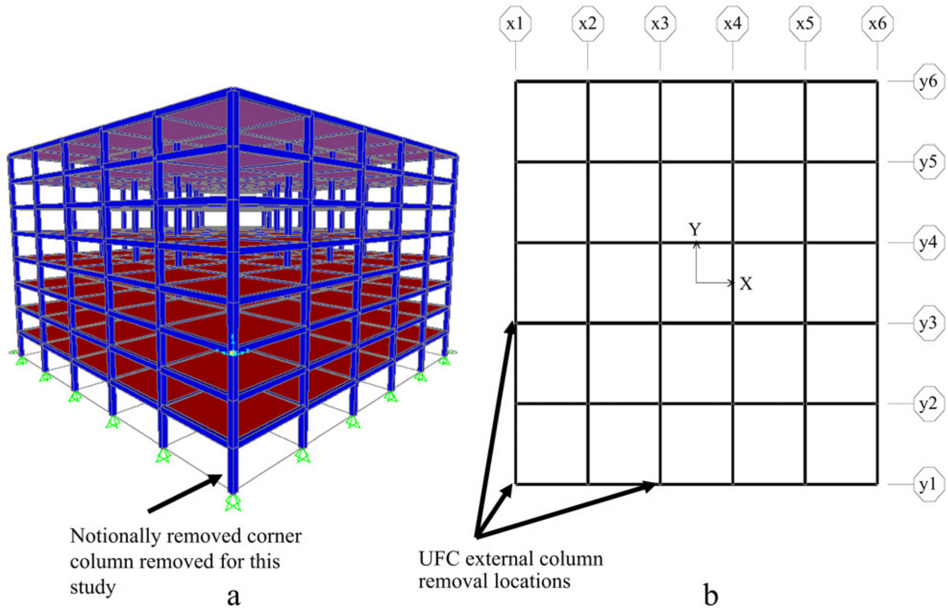

Fig. 1. (a) 8-story reinforced concrete column case study with corner column to be removed in AP investigation shown, (b) External columns removed for AP analysis to investigate potential of a framed structure for progressive collapse based on UFC [1].

Hansen [5] studied the performance of three-dimensional models of external columns in reinforced concrete buildings. The authors produced response histories for edge beams using nonlinear dynamic analysis to simulate the loss of exterior columns. The authors also demonstrated that nonlinear dynamic analysis is important for progressive collapse investigations to capture a realistic structural response.

Marjanishvili and Agnew [6] compared linear-static, linear-dynamic, nonlinear-static, and nonlinear-dynamic analysis methods for progressive collapse analysis based on the General Services Administration (GSA) provisions. The authors argued that GSA performance limits for linear analysis methods are not conservative and that linear-static and linear-dynamic analysis methods produce comparable maximum deformations.

Marchand [7] presented an extensive coverage of analysis and design methods for mitigation of the effects of progressive collapse and blast. Performance expectations in GSA and DoD guides are were also discussed in this paper.

Mohamed [8] pointed out the existence of torsion stresses at corner columns that are notionally removed during AP progressive collapse analyses. Torsion shear stresses may cause brittle beam failure and should therefore be removed from the model during AP analysis.

Kaewkulchai and Williamson [9] demonstrated that when dynamic analysis is used to assess the potential for progressive collapse of frames, the use of either the initial configuration or the deformed configuration does not significantly affect the structural response.

Iwankiew and Griffis [10] studied the collapse characteristics of various buildings in recent history. The authors argued that similar distress mechanisms are exhibited by steel and concrete buildings when collapse is caused by fire in combination with impact. They also pointed out that the architectural layout of the building could play significant in role in reducing or exacerbating the number of causalities caused by progressive collapse.

Grierson et al. [11] presented a method for conducting linear static progressive collapse analysis based on the provisions of the United States General Services Administration (GSA) [12]. The authors modeled the reduced stiffness during progressive collapse using and an equivalent-spring method.

2. The modeling process

In order to demonstrate the implementation of UFC, the stress and deformation demand resulting from the application of the AP method, a case study is modeled, analyzed, and designed. The following sections describe the modeling process.

2.1. Column heights

According to section UFC 2-2.1 and for all levels of protection, the laterally unsupported length of all columns used in the analysis and design should be equal to the height of two stories. This requirement is to ensure that each column is able to support vertical loads after loss of lateral support at any floor level. Column length to be removed for AP analysis is the clear length between lateral restraints. Engineers have interpreted section 2-2.1 to mean that the column unbraced length used in design should be equal to the height of two stories. However it is conservative to interpret the reference to *analysis* in section 2-2.1 to mean that columns equal to the height of two stories can be used to obtain column design forces and to design the columns.

2.2. Over-strength and strength reduction factors

A factor $\Omega = 1.25$ is applied to the concrete compressive strength, f_c', and to the yield strength of reinforcing steel, f_y, in the process of calculating the nominal flexural resistance, M_n, and shear resistance, V_n.

Strength reduction factors: The strength reduction factor, ϕ, for flexure, shear, axial, and torsion as specified in the ACI 318 code.

2.3. Strength and deformation limits

Structural elements and connections must meet the strength limits in Table 1 and deformation limits in UFC, Table 4-4 [2]. The deformation limits are listed in terms of maximum rotations for beams and slabs and in terms of ductility for columns. For HLOP and MLOP analyses, the maximum permissible rotation for reinforced concrete doubly reinforced and shear-reinforced beams when tension membrane is not included in the analysis is $\theta = 4°$.

Table 1
Strength limits from UFC 4-023-03, Tables 4-3.

Structural behavior	Acceptability criteria	Subsequent section for violation of criteria
Element flexure	ϕM_n	UFC 4-023-03 Section 4-3.1.1
Element combined axial and bending	ACI 318-02 Chapter 10 provisions	UFC 4-023-03 Section 4-3.1.3
Element shear	ϕV_n	UFC 4-023-03 Section 4-3.1.3
Deformation	Deformation limits, defined in UFC 4-023-03 Table 4-4	UFC 4-023-03 Section 4-3.2

A more generous maximum rotation of $\theta = 12°$ is permitted in HLOP and MLOP analyses for reinforced concrete beams when tension membrane is included in the analysis.

2.4. Analysis methods

Three analysis methods are explicitly permitted: Linear Static, Nonlinear Static, and Nonlinear Dynamic. Three-dimensional analysis is recommended but two-dimensional analysis is permitted if three-dimensional effects can be incorporated. The model presented in this paper shows the importance of incorporating three-dimensional effects, especially at the part of the structure where a column is notionally removed. The three analysis methods are discussed in the following paragraphs. However, a numerical case study will follow that uses linear static analysis. Linear static analysis procedures are the most commonly used in structural design professional practice compared nonlinear static and nonlinear dynamic methods. Furthermore, UFC describes a consistent procedure in which damage to structural elements and floor areas is considered. Linear static analysis should be used to assess the potential for progressive collapse of existing or proposed structures and not for predicting the post-yield behavior of the structures.

2.5. Linear static analysis

The assumptions for this analysis include small deformations (no geometric nonlinearity), and the material is considered linear elastic. The full load is applied at one time on the structure with the notionally removed element. The following design load combination is applied to bays immediately adjacent to the removed column and to all floors above the removed column:

$$2.0\,[(0.9D \text{ or } 1.2D) + (0.5\,L \text{ or } 0.2\,S)] + 0.2\,W \tag{1}$$

where:

D: Dead load.
L: Live load.
S: Snow load.
W: Wind load.

For the rest of the structure, the load combination is similar to the above but without the magnification factor of 2.0 applied to the gravity load portion of Eq. (1).

If the applied moment exceeds the nominal flexural strength at any location and the reinforcement layout allows a plastic hinge to form, place a discrete hinge to simulate the plastic at appropriate location within the element. At the discrete hinge location, two constant and opposite moments must be applied in a direction consistent with the original bending moment. The hinge should not be placed further than 1/2 the depth of the beam from the face of the column. The insertion of discrete hinges at locations where the nominal flexural strength is reached recognizes that the fact that if a potential hinge location is designed and detailed to have adequate rotation capacity it will continue to rotate under constant moment. The locations where discrete hinges are placed must be designed and detailed in order to have proper rotation capacity.

2.6. Nonlinear static

Both material and geometric nonlinearities are considered. The load history starts from zero to the full factored load. The design load combination in Eq. (1) is applied to bays immediately adjacent to the removed column and to all floors above the removed column.

Similar to Linear Static analysis, the rest of the structure is loaded with the combination in Eq. (1) but without the magnification factor of 2.0. During AP analysis, if a structural element can sustain constant moment during deformation and if the bending moment exceeds the flexural strength at any point, the analyst should manually insert a discrete hinge at that location. If the element cannot sustain constant moment during deformation, it is considered to have failed and the flexural strength is exceeded. The element must be removed from the model and its associated load redistributed. For nonlinear analysis, many structural analysis programs can determine hinge locations if the flexural strength is reached.

2.7. Nonlinear dynamic

Material and geometric nonlinearities are considered. The dynamic analysis is performed by instantaneous removal of a load-bearing element with the structure fully loaded. The load combination for the AP method for all construction types is:

$$(0.9D \text{ or } 1.2D) + (0.5\,L \text{ or } 0.2\,S) + 0.2\,W. \tag{2}$$

In this analysis method, both material and geometric nonlinearities are accounted for and the loads are applied dynamically.

With respect to shear strength, ϕV_n, any element that does not possess sufficient capacity is to be removed and the load redistributed, regardless of the analysis method. In certain circumstances, the shear failure can occur in such a way that the removal of the failed elements will cause area damage greater than the maximum permissible floor damage. This is the case with corner panels in symmetric buildings.

If the deformation limits prescribed in UFC are exceeded, the element should be removed from the model and the load redistributed.

2.8. Acceptable floor damage limits due to loss of a column

The UFC limits the permissible damage area due to the loss of an external column to the smaller of 70 m^2 (750 ft^2) or 15% of the total area of that floor. The document limits the permissible damage due to the loss of an internal column to the smaller of 140 m^2 (1500 ft^2) or 30% of the total area of that floor. Whether the damaged column is internal or external, the floor damage should not extend beyond the area tributary to the column or the bays immediately adjacent to the removed column.

3. Case study

To examine the requirements of UFC for progressive collapse mitigation of reinforced concrete buildings, a case study 8-story, cast-in-place building is used. The frame structure of the building,

Table 2
Gravity and loads applied to the case study reinforced concrete structure.

Dead load	Self weight	150
	Normal weight concrete (pcf)	
	Superimposed load uniform (psf)	30
	Superimposed load, cladding panels on perimeter beams (plf)	180
Live load		70
Wind load	Speed (mph)	100
	Exposure type	B
	Importance factor, I	1
	Topographical factor, k_{zt}	1
	Gust factor	0.85

Table 3
Dimensions and reinforcement of edge beams and typical columns in Model 1 at the ground floor level designed according to ACI 318-05.

		Dimensions (in. × in.)	Reinforcement (in.^{2})
Edge Beam Spans	Exterior	14 × 24	2.58 (Top) 1.24 (Bottom)
	Interior	14 × 22	2.37 (Top) 1.14 (Bottom)
Columns	Corner	22 × 22	4.84
	Exterior	22 × 22	8.75
	Interior	26 × 26	30.87

Table 4
Load cases created in SAP2000 to be combined based on Eq. (1).

Load case name	Factor load	Location of application
D09	0.9D	Panels not supported by the removed column
D09_2	2 × 0.9D	Panels supported by the removed column at all levels
D12	1.2D	Panels not supported by the removed column
D12_2	2 × 1.2D	Panels supported the removed column at all levels
L5	0.5 L	Panels not supported by the removed column
L5_2	2 × 0.5L	Panels supported the removed column at all levels
W02	0.2W	Windward and Leeward directions applied in two perpendicular axes.

shown in Fig. 1, does not contain irregularities in plan or elevation. All floors are 5-panels × 5-panels and each panel is 28-ft × 28-ft (8.5 m × 8.5 m). Dimensions are selected such that the area of corner panels is 784 ft^2 (72.8 m^2) which is greater than the 750 ft^2 (70 m^2) limit on floor damage prescribed in section 3-2.6.1 of UFC 4-023-03. Therefore, the structural elements supporting the corner floor panel must be designed such that the removal of a corner column on the ground floor does not lead to the failure of corner floor panel. The stresses on the supporting beams should not exceed the limits in Table 1 and UFC 4-023-3, Table 4-4. The case study will examine alternatives to prevent failure of the beams supporting the corner panel.

The structure is first analyzed and designed for gravity and wind forces. To investigate the potential for progressive collapse of this regular building due to notional removal of columns based on the AP method, a corner column at the ground floor level is removed as shown in Fig. 1. Although the AP analysis should be conducted at all floor levels, the concepts to be demonstrated in this paper will be illustrated by the most severe scenario, which is the removal of ground floor column. All analyses reported in this paper were conducted using SAP2000 program, developed by Computers and Structures, Inc [13]. The presence of steel bracing the stress and deformation demand during AP analysis is also examined.

3.1. Geometric model, material properties, and loads

Three models were created in this study. Model 1 represents the regular dimensions of the structure loaded with typical dead loads, live loads, and wind loads and designed according to the American Concrete Institute standard [1]. Models 2, 3, 4, and 5 are created to examine the progressive collapse potential of the structure in Model 1. Model 2 is created for AP analysis applied to a corner column that is part of a moment resisting frame. Models 3, 4, 5 are created for AP analysis applied to a corner column and represent structures with different configurations of steel bracing. The advantages and disadvantages of the models will be discussed. The selected service and wind loads are listed in Table 2.

3.1.1. Model 1

The structure in Fig. 1 is analyzed and designed based on this Model. The geometric properties of the cast-in-place reinforced concrete structure are listed in Table 3. For simplicity, all beams in exterior frames have the same dimensions and all columns in exterior frames have the same dimensions. The same applies to interior frames. Concrete compressive strength is, f'_c = 4000 psi (27.6 MPa) and steel grade is 60 (413.7 MPa).

The structure is analyzed and designed to meet ACI 318-05 [1] requirements. The required reinforcements for exterior and interior spans in the edge beam that contain the column that will be removed in subsequent models are listed in Table 3. The edge beam selected is at the ground floor level. The column that is notionally removed in Model 2, supports the exterior beam reported in Table 3.

In general, gravity and wind load combinations control the design of beams in the lower stories while gravity only load combinations control the design of beams in upper two stories.

All structural members met ACI 318-05 [1] design requirements.

3.1.2. Model 2

This model is used to examine the structure defined by Model 1 for progressive collapse potential due to the loss of a corner column. The area of the corner panel exceeds the limit of acceptable floor damage, therefore, it is imperative to examine the options to prevent the failure of beams supporting the corner floor panel when the supporting column is notionally removed. The process of placing a discrete hinge at the location of maximum bending moment prescribed by UFC for analysis methods works on interior panel. However for a corner panel, it is important to prevent flexural failure. A corner column is removed from the model as shown in Fig. 1.

The design load combination shown in Eq. (1) is applied to bays immediately adjacent to the removed column and to all floors above the removed column. For all other panels, the design load combination is the same as Eq. (2). Table 4 shows the load cases needed to generate the AP analysis load combination in SAP 2000.

In this model, column heights are increased from 12 ft to 24 ft to satisfy the requirements of UFC section 2-2.1. The deformed shape due to the load combination that produced the maximum stress demand (controlling load combination) is shown in Fig. 2.

The bending moment diagram for the controlling load case at the exterior frame, which contains the notionally removed column, is shown in Fig. 3. The removed column imposes significant bending moments at the edge beams and causes some stress reversal in adjacent beams. This may require top reinforcement at locations in the span where the main reinforcement would normally be at the bottom. The bending diagram on the top of Fig. 3 is for an interior frame not containing the notionally removed column which is provided here for comparison. Depending upon the spans, stress reversal may occur at many other beams as columns are removed in a complete AP analysis. ACI gravity load combinations used for strength design do not cause stress when the spans do not differ significantly and the ratio of live load to dead is small.

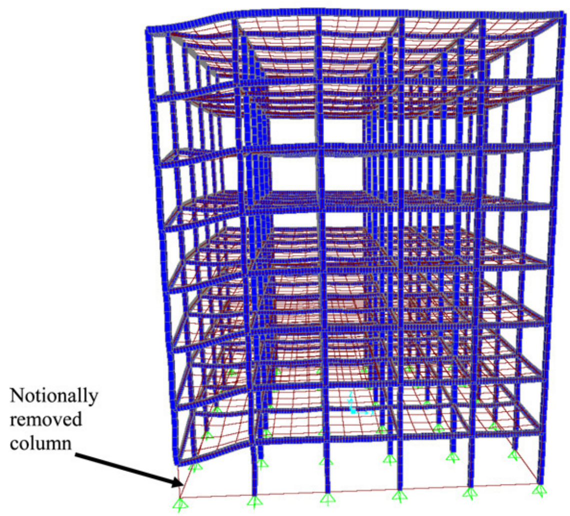

Fig. 2. Deformed shape of Model 2 subjected to controlling load combination.

Table 5 shows the dimensions of beams and columns that satisfy the requirements of Table 1 such as $V_u \leq \phi V_n$ and $M_u \leq \phi M_n$. Notice that interior beams maintained the same dimensions needed for Model 1 designed with the traditional load combinations because the progressive collapse potential of interior floor panels is not included in this study. However, exterior beam depths needed to be increased from 24 in. (61 cm) to 28 in. (71 cm), which amounts to (+16.6%). The spans selected in this study cause the corner panels to barely exceed the floor damage limits prescribed. Much longer spans would require deeper beams.

Table 5
Dimensions and reinforcement of edge beams and typical columns in Model 2 at the ground floor level designed according to ACI 318-05.

		Dimensions (in. × in.)	Reinforcement (in.^{2})
Edge beam spans	Exterior	14 × 28	8.16 (Top) 3.71(Bottom)
	Interior	14 × 22	3.00 (Top) 1.44 (Bottom)
Columns	Corner	22 × 22	4.84
	Exterior	22 × 22	22.94
	Interior	26 × 26	16.81

The required flexural reinforcements for the spans of the edge beam that contains the removed column at the ground level are listed in Table 5. Notice that Table 5 includes the required reinforcement for the current AP analysis that uses Model 2 and not the final design reinforcement, which will have to include all models and load cases.

Interior column reinforcement is lower than exterior column because the design load combination includes higher gravity loads in the corner panel than interior panels, as shown in Eq. (1). ACI 318 load combinations for the strength design method control the design of interior columns for this case study.

In comparison with Model 1, which uses the traditional ACI 318-05 load combination, Model 2 requires more flexural

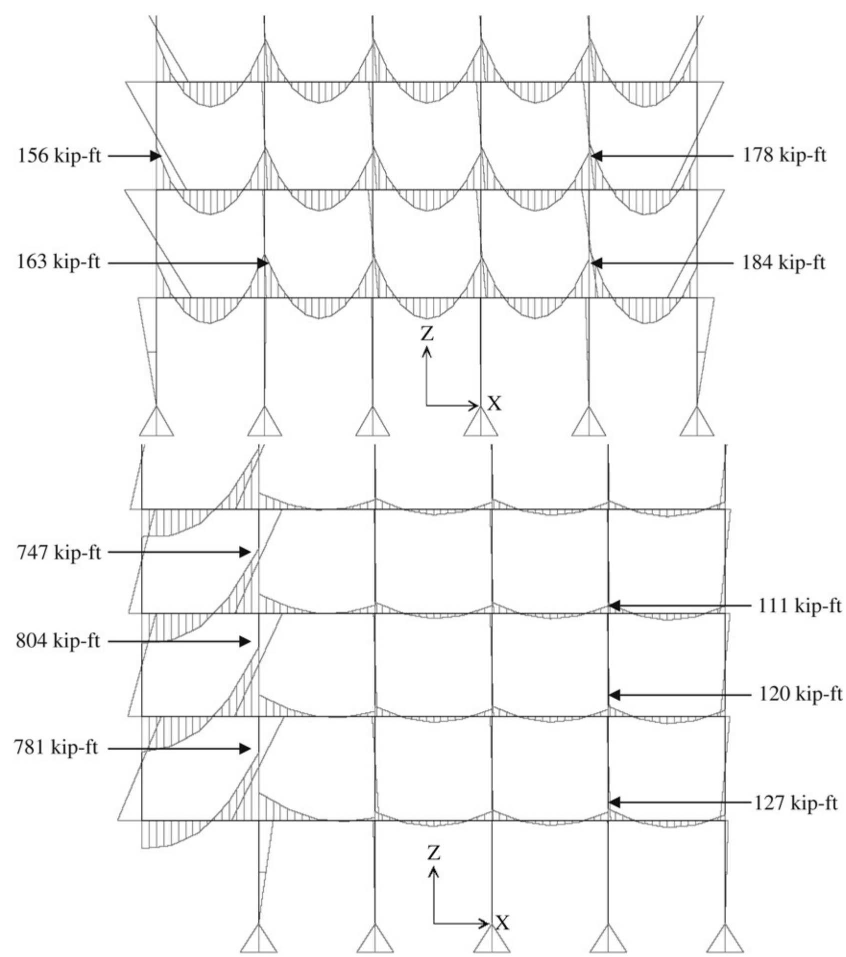

Fig. 3. Bending moment diagram for the exterior frame containing the notionally removed column (bottom) and a typical interior frame not containing the removed column (top), for Model 2.

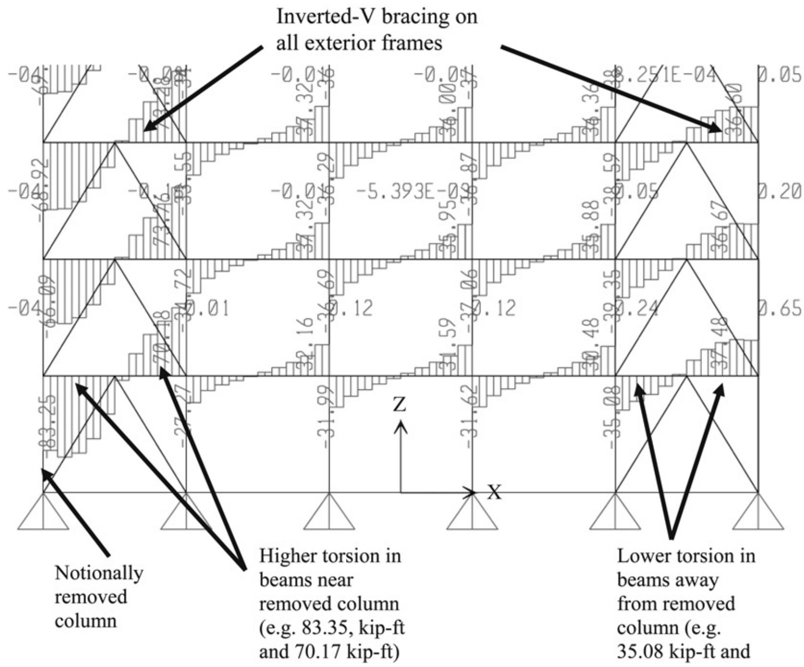

Fig. 4. Inverted-V bracing at exterior frame for lateral load resistance (lower floors shown) with torsion diagram due to controlling UFC load combination for Model 3.

reinforcement and greater depth at the edge beam. This is due to the long cantilevers created by the notional removal of the corner column. However, columns in Model 2 required less reinforcement with the same dimensions as Model 1. This is because the load factors in the AP analysis used in Model 2 are smaller than the traditional load factors that ACI 318 requires for Model 1. It is therefore important for structural designers performing progressive collapse design in addition to the traditional ACI design to realize that come structural elements in the same building may be controlled by ACI load combinations while other UFC load combination for AP analysis.

3.1.3. Model 3

Model 2 showed that AP analysis applied to corner columns imposes significant flexural demand on beams supporting corner panels. The case study in this paper uses beam spans that make corner panels only slightly above the damage limits in UFC. One option is to support or reduce the spans using steel bracing. Steel bracing may be used to resist lateral forces and reduce lateral deformations. The same bracing, if present, can reduce the span as shown in this model, and at the same time pick up some of the axial forces generated by the removal of the corner column. For this model, the dimensions of beams are the same as in Model 1 in order to investigate the effect of bracing on flexural demand, but the loads and story heights are similar to those in Model 2 in order to investigate progressive collapse of the corner panel. Fig. 4 shows typical inverted V-bracing used on all exterior frames. All members of the inverted V-bracing are W24 × 131. The material of the bracing system in Models 3, 4, and 5 is A36 steel. The frame shown includes the notionally removed column.

The flexural strength of all elements satisfied the requirements of Table 1, that is $M_u \leq \phi M_n$. The flexural reinforcement summary is shown in Table 6. Clearly flexural demand on beams and axial reinforcement in columns decreases when bracing is added. The benefits of bracing to flexural reinforcement demand is more pronounced if beams spans were longer.

Despite the reduction in flexural demand, sixteen elements failed due to the high shear forces generated in beams supporting the panels above the notionally removed columns. The beams that failed are the ones supporting the corner floor panels above the notionally removed column. The failure is due the high torsion generated in the exterior beams supporting these vulnerable panels at all levels above the notionally removed column.

Table 6
Dimensions and reinforcement of edge beams and typical columns in Model 3 at the ground floor level designed according to ACI 318-05.

		Dimensions (in. × in.)	Reinforcement (in.^{2})
Edge beam spans	Exterior	14 × 28	3.11 (Top) 1.45 (Bottom)
	Interior	14 × 22	1.21 (Top) 0.84 (Bottom)
Columns	Corner	22 × 22	4.84
	Exterior	22 × 22	11.94
	Interior	26 × 26	15.74

3.1.4. Model 4

This model is similar to Model 3, except that the bracing configuration is changed, as shown in Fig. 5a, to examine the effect of bracing configuration on flexural demand. In this case, the bracing is under compression in the controlling load combination because gravity loads are dominant. All members of the bracing system in this Model are W36 × 300. The bending moment diagram for the controlling gravity load case is shown in Fig. 5b and the corresponding shear force diagram is shown in Fig. 5c.

Twelve beams failed in comparison with sixteen for the bracing configuration in Model 3. However, similar to Model 3, the failing beams are due to excessive torsion-induced shear stress. The beams that failed are supporting the floor panels above the notionally removed columns. Also, similar to Model 3, the flexural demand is lower than Model 2 because the steel bracing provides additional support to the corner beams. The reinforcements needed to meet to flexural demand in this model are shown in Table 7.

3.1.5. Model 5

This model is similar to Models 3 and 4, except that the bracing configuration is changed as shown in Fig. 6a. In this case the bracing

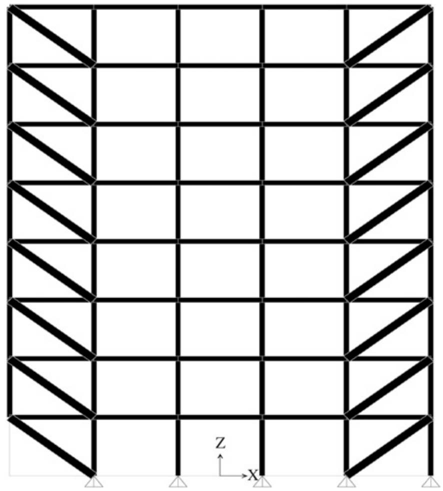

Fig. 5a. Steel bracing to external frames oriented such that they are under compression due to gravity load and used in Model 4.

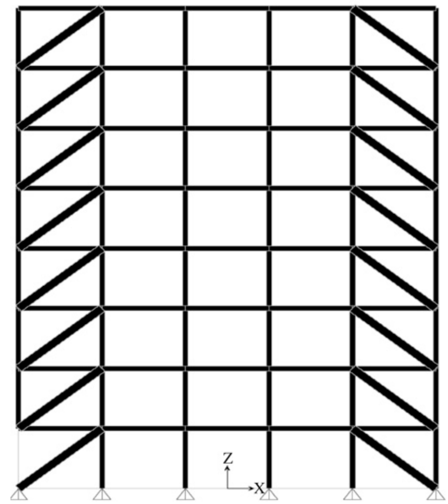

Fig. 6a. Steel bracing to external frames oriented such that they are under tension due to gravity loads used in Model 5.

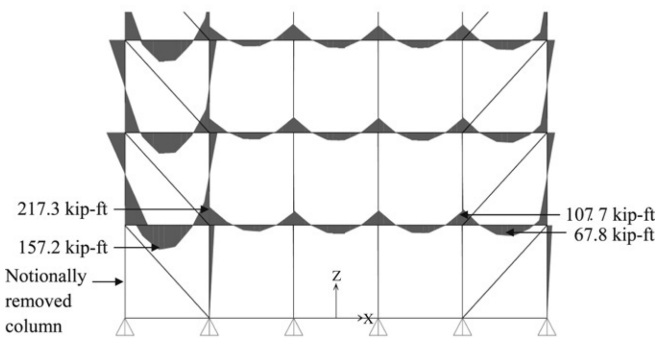

Fig. 5b. Bending moment diagram for the braced frame in Model 4. Bending moments are higher in the panels supported by the removed column.

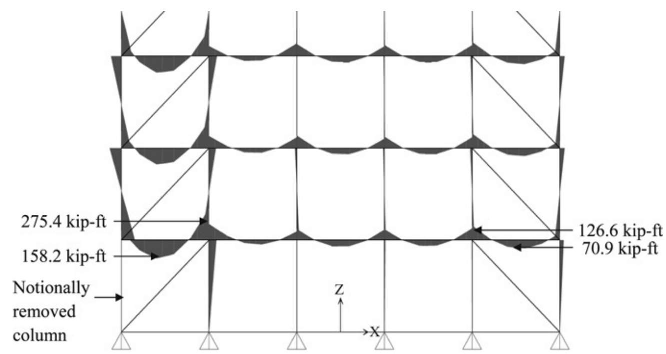

Fig. 6b. Bending moment diagram for the braced frame in Model 5. Bending moments are higher in the panels supported by the removed column.

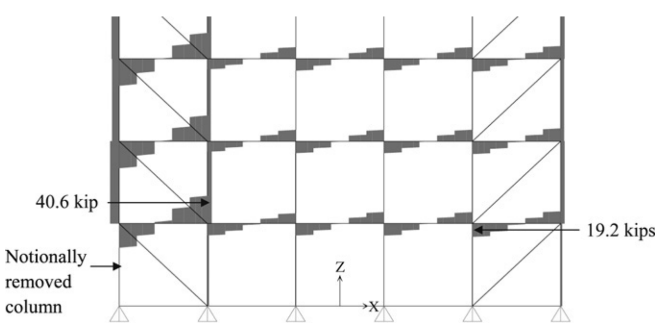

Fig. 5c. Shear force diagram for the braced frame in Model 4. Higher shear forces develop in the panels supported by the notionally removed column.

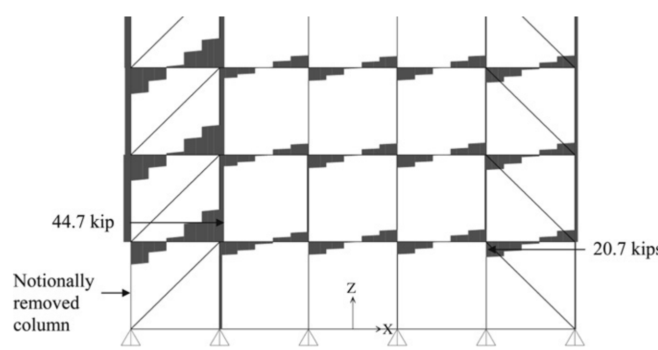

Fig. 6c. Shear force diagram for the braced frame in Model 5. Higher shear forces develop in the panels supported by the notionally removed column.

Table 7
Flexural reinforcement required for edge beams and columns in Model 4 to meet ACI318-05 requirements.

Required reinforcement (in.^{2})						
Edge beams (in.^{2})				Columns (in.^{2})		
Exterior		Interior		Corner	Exterior	Interior
Top	Bottom	Top	Bottom			
2.37	1.63	1.26	0.83	4.84	4.84	6.76

elements are in tension due to the dominant gravity forces. All members of the bracing system in this Model are W36 $\times$ 300. The bending moment diagram for the controlling gravity load case is shown in Fig. 6b and the corresponding shear force diagram is shown in Fig. 6c.

Twelve beams failed in shear, similar to Model 4. The beams that failed are the ones supporting the floor panels above the notionally removed column. The flexural reinforcement for the edge beams at the ground floor level is lower than Model 2. Flexural reinforcement for edge beams and columns are shown in Table 8. Similar to Models 3 and 4, flexural demand is lower than in Model 2 because of the additional support provided by the reinforcement.

The difference in flexural demand between Model 4 and Model 5 is not significant but the size of bracing members is smaller.

Table 8
Flexural reinforcement required for edge beams and columns in Model 5 to meet ACI318-05 requirements.

Required reinforcement (in.^{2})						
Edge beams (in.^{2})				Columns (in.^{2})		
Exterior		Interior		Corner	Exterior	Interior
Top	Bottom	Top	Bottom			
3.05	1.57	1.48	0.97	4.84	4.84	6.76

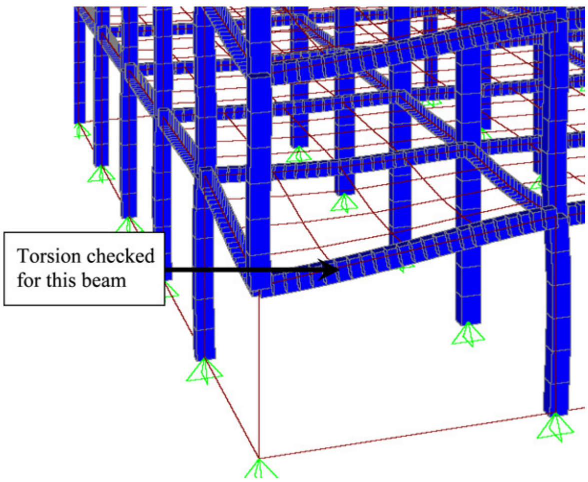

Fig. 7. One of the beams at the ground floor framing into the notionally removed column.

3.1.6. Model comparison and torsional shear failure investigation

A better understanding of the structural response in an AP analysis of Models 3 through 5 is obtained by examining the forces in one of the two beams framing into the notionally removed column at the ground floor level. Maximum bending moment, shear force, torsion, and axial in the ground floor edge beam, regardless of the location within the span are shown in Table 9.

Model 2 experiences the largest design bending moment of all four models because of the long unsupported cantilevers. All members in this model met ACI 318-05 design requirements when exterior beam depths are increased to 28 inches. Bending moments in Models 3 through 5, which are all braced frames, are significantly lower than Model 2; therefore, the depth needed to resist flexure is 24 inches (61 cm). However, a depth of 24 inches (61 cm) is not sufficient to resist shear due to torsion, especially with v-shaped bracing used in Model 3 where 16 beams fails in shear caused by torsion. The reason for shear failure for most of the beams is crushing of concrete structures as demonstrated in the calculations related to the beam in Fig. 7. The table above also shows that axial loads on the beams of Model 2 are much smaller than Models 3, 4, and 5. The highest axial force occurs in the beams of Model 4. Axial force helps increase the cracking torsion resistance.

The beam shown in Fig. 7 failed is one of the 16 beams that failed due to torsion-induced shear in Model 2. Analysis is show shown below:

$Tu = 76.85$ kip ft (104.2 kN m). The critical section is assumed at a distance an-effective-depth from the face of the column.

$Vu = 16.52$ kips (73.5 kN). Shear force at the location of critical torsion moment.

$Nu = 34.5$ kips (153.5 kN), compression.

Area enclosed by outside perimeter of concrete cross-section.

$A_{cp} = 336$ in.^{2} (2167.7 cm^2).

Outside perimeter of concrete cross-section.

$P_{cp} = 76$ in.^{2} (490.32 cm^2).

Cracking torque for nonprestressed members subjected to an applied axial force:

$$\phi T_{cr} = 0.75\left(4\sqrt{f_c'}\left(A_{cp}^2/P_{cp}\right)\sqrt{1+N_u/\left(4A_g\sqrt{f_c'}\right)}\right)$$
$$= 27.88 \text{ kip ft} = 37.92 \text{ kN m}$$

$T_u > \phi T_{cr}/4$, therefore, design for torsion is required according to section 11.6.1 of ACI 318-05.

Assuming 1.75 inches (4.44 cm) cover to the center line of stirrups

$$A_{0h} = (14 - 2 \times 1.75) \times (24 - 2 \times 1.75)$$
$$= 10.5 \times 20.5 = 215.25 \text{ in.}^2 \ (1388.7 \text{ cm}^2)$$

$$A_0 = 0.85A_{0h} = 0.85(247.25) = 183 \text{ in.}^2 \ (1180.6 \text{ cm}^2)$$

$$A_t/s = T_u/\left(2\phi A_o f_{yv}\cot\theta\right)$$

$$A_t/s = 76.85 \text{ kip ft}(12000)/\ (2 \times (0.75)(183)(60,000)\cot(45))$$

$$A_t/s = 0.056 \text{ in.}^2/\text{in/leg}(1.42 \text{ mm}^2/\text{mm/leg})$$

$$\phi V_c = \phi 2\sqrt{f_c'}b_w d = (0.95)2\sqrt{4000}(14)(22.5)$$
$$= 30.12 \text{ kips } (133.97 \text{ kN})$$

$0.5 < \phi V_c < V_u < \phi V_c$.

Minimum shear reinforcement

$$A_v/s = 0.75\sqrt{f_c'}b_w/f_{yt} = 0.01106 \text{ in.}^2/\text{in.} \ (0.28 \text{ mm}^2/\text{mm}).$$

Shear reinforcement should not be less than

$$A_v/s = 50b_w/f_{yt}$$

$$A_v/s = 0.01167 \text{ in.}^2/\text{in.} \ (0.3 \text{ mm}^2/\text{mm}).$$

Combined shear and torsion stirrup requirements

$$At/s + Av/2s = 0.056 + 0.001106/2$$
$$= 0.0615 \text{ in.}^2/\text{in/leg } (1.56 \text{ mm}^2/\text{mm/leg}).$$

Using # 4 reinforcement.

$s = 0.2/0.0615 = 3.25$ in. (82.6 mm).

Check for crushing of the concrete compression struts

$P_h = 2 \times 10.5 + 2 \times 20.5 = 62$ in.(1574.80 mm)

$$\sqrt{\left(V_u/\left(b_w d\right)\right)^2 + \left(T_u P_h/\left(1.7A_{0h}^2\right)\right)^2} \le \phi\left(V_c/\left(b_w d\right) + 8\sqrt{f_c'}\right)$$
$$= \phi 10\sqrt{f_c'}$$

$$\sqrt{\left(16520/\left(14 \times 22.5\right)\right)^2 + \left(76.85 \times 12000 \times 62/\left(1.7 \times 215.25^2\right)\right)^2}$$
$$= 727.8\text{psi}$$

$$\phi 10\sqrt{f_c'} = 0.75(10)\sqrt{4000} = 474.34\text{psi}$$

$$\sqrt{\left(V_u/\left(b_w d\right)\right)^2 + \left(T_u P_h/\left(1.7A_{0h}^2\right)\right)^2} > \phi 10\sqrt{f_c'}.$$

Table 9
Bending moment–shear force–torsion–axial force imposed on an edge beams supporting the notionally removed corner column in Models 2, 3, 4, and 5.

Force	Model 2	Model 3	Model 4	Model 5
Bending moment, M_u (kip ft)	−776.96	−291.28	−217.31	−275.32
Shear force, V_u (kips)	80.43	64.14	40.64	44.73
Torsion, T_u (kip ft)	62.33	76.85	53.49	49.94
Axial force, N_u (kips)	−9.76	−34.68	148.89	−91.08

The beam fails by crushing of the concrete compression struts. Therefore, when a corner column is removed for AP analysis, shear failure caused by combination of direct shear and torsion may control the design of edge beams in the spans adjacent to the notionally removed column. Fig. 4 shows that torsion decreases away from the notionally removed corner column. To reduce shear stresses such that ACI 318 is provisions are satisfied, the sizes of these beams will have to be increased. When removing corner columns, shear induced torsion is the controlling criterion in AP analysis. However, flexure is the controlling criterion when internal columns are removed.

4. Summary and conclusions

- The design of corner floor panels against progressive collapse based on DoD guidelines and using linear elastic analysis poses high flexural demands on beams compared with load combinations used in the strength design method of ACI 318. This is due to doubling of gravity loads on the corner panel that includes the notionally removed column and all the panels above it as required by the DoD design guide. It is also due to long cantilevering beams created by the notional removal of the corner panel.
- Three-dimensional analysis is especially important to account for torsional shear stresses at the part of the structure where the column is notionally removed in the AP analysis. Shear stresses due to torsion can control the design.
- Models examined in this study include braced frames and deep edge frames. Systems with bracing can divert some of the high forces generated by the removed column due to an AP analysis to the bracing system, if the connections are designed to support and transfer additional loads. However, high torsion-induced shear forces are generated that can lead to shear failure. Systems with moment resisting frames, on the other hand, experience high flexural stresses for which the structural elements must be designed.

References

[1] ACI-318. Building code requirements for structural concrete (ACI318 –05) and commentary (ACI 318R05). Detroit (MI): American Concrete Institute, 2005.
[2] Design of structures to resist progressive collapse – UFC 4-023-03. Washington (DC): United States Department of Defense, 2005.
[3] Mohamed OA. Progressive collapse of structures: Annotated bibliography and comparison of codes and standards. J Perform Const Facil 2006;20(4): 418–425.
[4] Powel G. Progressive collapse: Case studies using nonlinear analysis. In: Proceedings of metropolis and beyond-structures congress, Reston (VA): ASCE, 2005.
[5] Hansen E, Wong F, Lawver D, Oneto R, Tennant D, Ettouney M. Development of an analytical database to support a fast running progressive collapse assessment tool. In: Proceedings of metropolis and beyond-structures congress, Reston (VA): ASCE, 2005.
[6] Marjanishvili S, Angew E. Comparison of procedures for progressive collapse analysis. J Perform Const Facil 2006;20(4):364–74.
[7] Marchand KA. Facts for steel buildings: Blast and progressive collapse. Chicago (IL): American Institute of steel construction, 2005.
[8] Mohamed OA. Investigating the capacity of reinforced concrete framed structures to sustain loss of primary load-carrying members. In: Proceedings of 11th international conference on civil, structural and environmental engineering computing. 2005.
[9] Kaewkulchai G, Williamson E. Dynamic behavior of planar frames during progressive collapse. In: Proceedings of 16th engineering mechanics conference. Reston (VA): ASCE; 2003.
[10] Iwankiew N, Griffis L. Comparison of structural performance of multi-story buildings under extreme events. Chicago (IL): American Institute of Steel Construction; 2004.
[11] Grierson D, Safi M, Xu L, Liu Y. Simplified methods for progressive-collapse analysis of buildings. In: Proceedings of metropolis and beyond – structures congress. Reston (VA): ASCE; 2005.
[12] United states general services administration. Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects. Washington, DC; 2003.
[13] SAP2000 software. Computers and structures-Inc. Berkeley, CA.