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Calculation of load increase factors for assessment of progressive collapse potential in framed steel structures

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ABSTRACT

Progressive collapse of building structures is a relatively rare event. However, the consequences of progressive collapse may be catastrophic in terms of injuries and loss of lives. In addition, in many parts of the world including the United States of America, Europe, Asia, and recently, United Arab Emirates, there is a trend to build taller and more structurally complicated buildings with adventurous load paths. Therefore, structural design that takes into account the potential for progressive collapse is becoming critical. This paper outlines and discusses the process of estimating the load increase factor (LIF) needed for progressive collapse resistant design of steel building structures that takes into account the effects of component ductility on structural response following the initiation of collapse. LIF are used to account for the dynamic effects of column/wall removal when the designer opts for linear or nonlinear static analysis to assess the potential for progressive collapse. The approach recognizes the difference in response associated with deformation-controlled compared to force-controlled response quantities and structural elements. Emphasis in this paper is on the Alternate Path (AP) approach which is the most commonly used approaches for progressive collapse resistant design of building structure that fall under Occupancy Category II.

Introduction

Assessment of the potential for progressive collapse of a building structure takes place after the design of all structural elements and components is completed in accordance with the applicable building codes in a country or region. The goal of progressive collapse assessment is to determine the capability of the structural system and components to transfer structural loads following the loss or significant damage of a primary load-carrying element. Components found deficient are redesigned to satisfy collapse mitigation requirements. The AP approach discussed in this paper for assessment of progressive collapse potential is similar in many ways to procedures described in various provisions/standards. However, this paper emphasizes the United States Department of Defense progressive collapse design provisions contained in UFC 4-023-03 [1]. As a minimum, vertical elements are removed for AP investigations at: 1st story above grade, story directly below roof, story at mid-height of building, and a story above the location of a column splice or where change in column size occurs.

For each of the stories indicated above, the entire framed structure is assessed for progressive collapse potential when critical external and internal columns are notionally removed. This paper focusses on performance of the structural system following the loss of corner columns, in particular. Corner columns are particularly vulnerable as practical structures rarely have the ability to span unsupported for long distances to transfer loads to other elements [2]. Each structural element,

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Nomenclature

$\Phi\ Rn$	design strength
Φ	strength reduction factor
Rn	nominal strength
$Ru = \Sigma\gamma i\ Qi$	required strength
γi	load factor
Qi	load effect
Q_{CE}	expected strength of the component or element
Q_{UDLim}	internal forces and moments
G_{LD}	increased gravity loads for deformation-controlled actions calculated using linear static analysis procedures
D	dead load including façade loads (kN/m^2)
L	live load including live load reduction (kN/m^2)
S	snow load (kN/m^2)
d	overall depth of beam
d_{bg}	depth of bolt group
Ω_{LD}	load increase factor for calculating deformation-controlled actions for linear static analysis
G_{LF}	increased gravity loads for force-controlled actions for linear static analysis
Ω_{LF}	load increase factor for calculating force-controlled actions for linear static analysis
Q_{UD}	deformation-controlled action
m	component or element demand modifier (m-factor)
Φ	strength reduction factor for the action considered
Q_{CE}	expected strength of the component or element for deformation-controlled actions
Q_{UF}	force-controlled action, from linear static model
Q_{CL}	lower-bound strength of a component or element for force-controlled actions

primary or secondary, must be assessed and designed to achieve design goal such as Immediate Occupancy (IO), life safety (LS), collapse prevention (CP), etc. These performance levels are the same as those defined in ASCE 41 [3].

For progressive collapse resistant design, each steel component and connections must satisfy Load and Resistance Factor Design (LRFD) Eq. (1).

$$\Phi R_n \geq R_u \tag{1}$$

The design strength $\Phi\ Rn$ is calculated using AISC LRFD [4]. The required strength, also known as *the actions*, $\Sigma\gamma i\ Qi$, may be determined using linear or non-linear analyses as appropriate. For the purposes of calculating component capacity, *actions*, such as bending moments or shear forces, are classified as either *deformation-controlled*, or *force-controlled*. Therefore, a component may need to be checked for both deformation-controlled and force-controlled actions. Typical *deformation-controlled* or *force-controlled*, classifications are shown in Fig. 1 [1]. Primary component action is *deformation-controlled* if it has a Type 1 curve and e ⩾ 2g, or, it has a Type 2 curve and e ⩾ 2g.

Define a primary component action as force-controlled if it has a Type 1 or Type 2 curve and e < 2g, or, if it has a Type 3 curve.

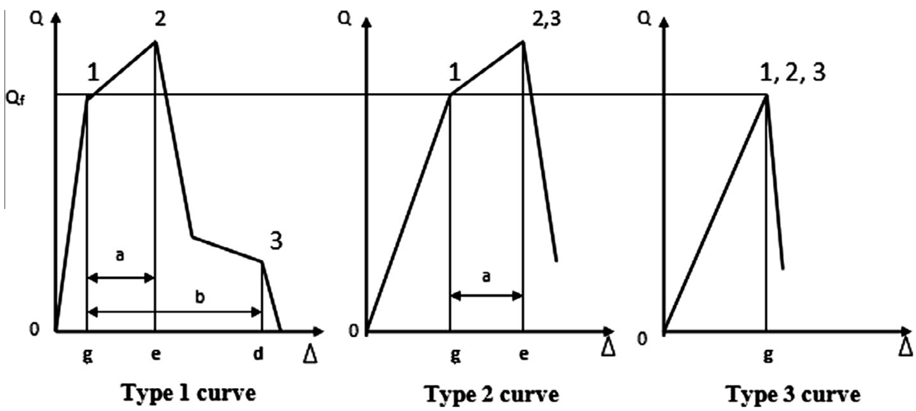

Fig. 1. Force-deformation curves for classification of actions as tension-controlled or.

UFC 4-023-03 [1] derives the definitions of deformation-controlled and force-controlled components from ASCE 41 [3]. For example, in moment-resisting-frames, bending moments in beams and columns are considered deformation-controlled while shear and axial forces are considered force-controlled.

Modeling and analysis procedures

The discussion in this paper is limited to mitigation of progressive collapse in structures that meet the requirements for the use of linear or nonlinear static analysis procedures. A case study is presented in this paper that demonstrates the process using Linear Static Procedures. LSP is permitted for regular or irregular structures where the Demand–Capacity Ratios (DCRs), as defined in Eq. (2), for each component does not exceed 2.00.

$$DCR = Q_{UDLim}/Q_{CE} \tag{2}$$

LSP is also permitted, regardless of DCR, when the structure does not contain any of the irregularities described in section 3-2.11.1.1 of UFC 4-023-03 [1].

Design load for deformation-controlled actions Q_{UD} and force-controlled action Q_{UF}

Three dimensional models are required to determine the deformation-controlled and force-controlled actions. As described below, separate computer models are needed to determine the deformation controlled-action Q_{UD} and the force controlled-actions, Q_{UF}.

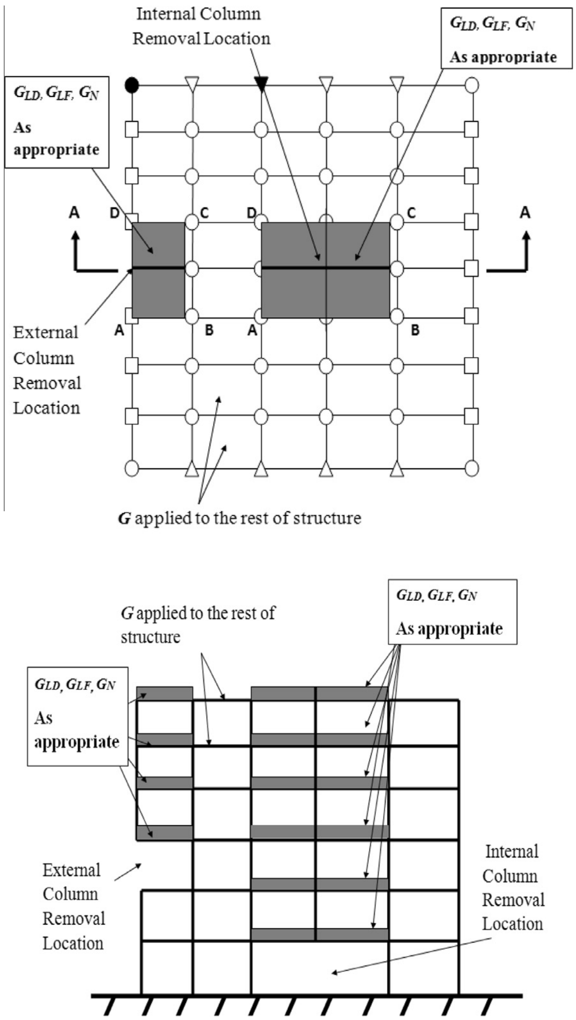

Fig. 2. Gravity load combinations on bays adjacent to the removed column and bays away from the removed column.

Load combinations to determine deformation-controlled actions, Q_{UD}

The design load combination to determine deformation-controlled actions using three dimensional computer models are discussed in this section and shown graphically in Fig. 2. Eq. (3) represents the gravity load combination applied to those bays immediately adjacent to the removed column and at all floors above the notionally removed column. The magnification factor G_{LD} is used to account for dynamic effects of column loss, when LSP is used to determine deformation-controlled actions.

$$G_{LD} = \Omega_{LD}[1.2\,D + (0.5\,L \text{ or } 0.2\,S)] \tag{3}$$

Eq. (4) represents the gravity load combination that is applied to those bays not loaded with G_{LD}. Unlike Eq. (3), this combination does not including dynamic magnification.

$$G = 1.2\,D + (0.5\,L \text{ or } 0.2\,S) \tag{4}$$

As shown in Fig. 2, the gravity load without magnification is applied at floor panels not adjacent to the notionally removed column.

External load on model to determine force-controlled actions Q_{UF}

This section describes the forces applied to the three-dimensional model to calculate the force-controlled actions. Eq. (5) represents the magnified gravity load combination that must be applied to those bays immediately adjacent to the removed element and at all floors above the removed element.

$$G_{LF} = \Omega_{LF}[1.2\,D + (0.5\,L \text{ or } 0.2\,S)] \tag{5}$$

Eq. (6) represents gravity load combination to be applied at floor areas away from removed column.

$$G = 1.2\,D + (0.5\,L \text{ or } 0.2\,S) \tag{6}$$

Load increase factors Ω_{LF} and Ω_{LD}

The load increase factor for force-controlled actions in framed steel structures is $\Omega_{LF} = 2.0$. For deformation-controlled, the load increase factor is given by Eq. (7):

$$\Omega_{LD} = 0.9\,m_{LIF} + 1.1 \tag{7}$$

m_{LIF} is the smallest component demand modifier, m-value, of all primary beam, girder, or spandrel that is connected to the columns directly above the column removal location. Typical m-factor is show in Table 1 [3]. Although Table requires input in U.S. customary units, the m-factors are dimensionless. Before m-factor for an element or connection can be determined, it must be classified as primary or secondary in terms of its contribution resistance of progressive collapse. Structural elements and components that provide the capacity of the structure to resist collapse subsequent to removal of a vertical load-bearing element are designated as primary; otherwise they are secondary elements/components [1].

Checking suitability of element/connection for deformation-controlled actions

For deformation-controlled actions, all primary and secondary components must satisfy Eq. (8).

$$\Phi\, m\, Q_{CE} \geq Q_{UD} \tag{8}$$

Table 1
m-Factors for various connection types in LSP.

Component	m-Factors for linear procedures				
	Primary component			Secondary component	
	IO (Immediate Occupancy)	LS (life safety)	CP (collapse prevention)	LS	CP
Beams – flexure					
a. $\frac{b_f}{2t_f} \leqslant \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leqslant \frac{418}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \geqslant \frac{65}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \geqslant \frac{418}{\sqrt{F_{ye}}}$	1.25	2	3	3	4
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				
Welded cover plate in WUF	$3.9 - 0.059d$	$4.3 - 0.083d$	$5.4 - 0.090d$	$5.4 - 0.090d$	$6.9 - 0.118d$
Improved WUF – bolted web	$2.0 - 0.016d$	$2.3 - 0.021d$	$3.1 - 0.032d$	$4.9 - 0.048d$	$6.2 - 0.065d$
Improved WUF – welded web	3.1	4.2	5.3	5.3	6.7
Simple shear tab		$5.8 - 0.107d_{bg}$		$8.7 - 0.161d_{bg}$	

The calculation of the expected strength, Q_{CE}, is delegated fully to ASCE 41 [3], confirming that the expected strength for progressive collapse resistance is assumed identical to that expected for seismic resistance. The m-factor accounts for expected ductility associated with deformation-controlled actions at specific or desired performance level. Performance levels may include life safety, collapse prevention, etc. The *Φ-factor*, however is identical to the values in applicable standards for the specific building material [4]. Discussion on the calculation of Q_{CE} is beyond the scope of this paper.

Checking suitability of element/connection for force-controlled actions

For force-controlled actions in all primary and secondary components must satisfy Eq. (9).

$$\Phi\, Q_{CL} \geq Q_{UF} \tag{9}$$

Q_{CL}, the lower-bound strength, shall be determined by considering all coexisting actions on the component under the design loading condition by procedures specified in ASCE 41 [3].

Case study

In order to demonstrate the calculation of the magnification factor, Ω_{LD}, the structural elements must be realistic. The following case study building structure was designed using the International Building Code (IBC) [5] in order to determine beam and column sizes. The software ETABS (CSI America Inc. USA) was used to create the model.

Dimensions and properties

Structure is regular with 5 × 3 panels on plan view and all spans are 9.00 m on centers in each direction, as shown in Fig. 3. Moment-frames were used on all perimeters with Improved Welded Unreinforced Flange (WUF) with bolted web. Connections at members other than perimeter moment resisting frames are flexible shear tabs.

The members were sized for reducible Live load (including allowance for partitions) of 4.5 kN/m^2, perimeter cladding load of 3.2 kN/m.

Wind load parameters were obtained in accordance with ASCE-10 [6]. Wind load was assumed to control for lateral load combinations. Assumed exposure category for wind analysis is B, design wind speed is 115 miles per hour (161 km/h), and building Occupancy Category II.

Floor System consists of 200 mm deep, normal weight reinforced concrete slab with 30 MPa strength.

Steel framing members: ASTM A992 grade 50 W-shapes

Flexible shear tab connections: 9.5 mm (3/8 in) plate with 4 19 mm (3/4 in) A490 N bolts. Depth of bolt group, d_{bg} = 228 -mm (9 in).

Analysis and design

An analysis was conducted using AISC *direct analysis* method and second-order effects were accounted for using AISC [4] general 2nd order. Moment resisting frames were used at the perimeters and gravity beams are used internally. Gravity beams were assumed in the structural model to be simply supported shear tabs.

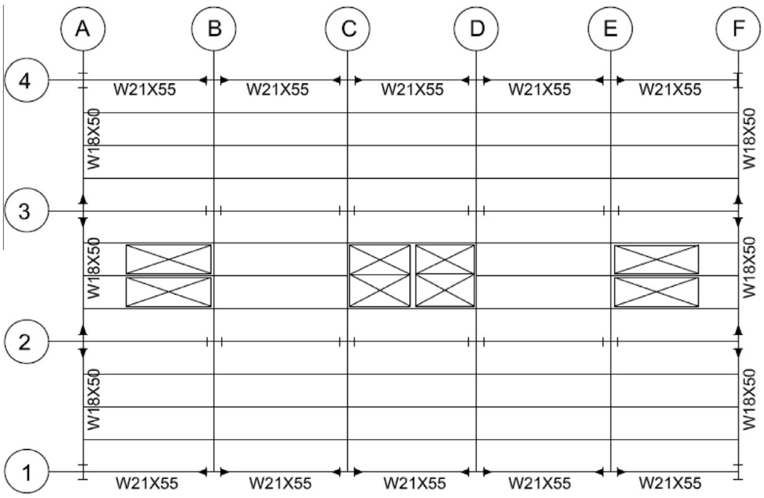

Fig. 3. First floor plan of 10-story steel framed building with perimeter moment frame and Improved WUF moment connections.

After analysis and design all members passed the AISC LRFD design requirements [4]. The controlling load combination for column A1 which is part of the moment frame along grid-lines 1 and A is 1.2D + 0.5 L + W. Fig. 4a shows the moment diagram for the controlling load combination of column A1, which includes an envelope of positive and negative values of wind-induced moments. Fig. 4b shows the demand/capacity (D/C) ratios for all columns in the moment frame along grid line A. It is worthy to note that structural elements were not all controlled by the same D/C ratio. The choice of corner column A1 is made due to the critical nature of this column [2], where the structure does not have an alternate load path to redistribute the loads safely. This is particulary true when the spans are relatively long, as the case in this study.

Column A1 connecting the base to the first floor level was chosen to demonstrate the process of calculating the magnified load factor. For this purpose, we identify the elements near the notionally removed column as:

Composite beam A2-B2: W18 x 50	Shear-tab connection at each end
Composite beam B1-B2: W24 x 62	Shear-tab connection at each end
Non-composite beam A1-A2:W18 x 50	moment resisting connection at both ends
Non-composite beam A1-B1:W21 x 55	moment resisting connection at both ends

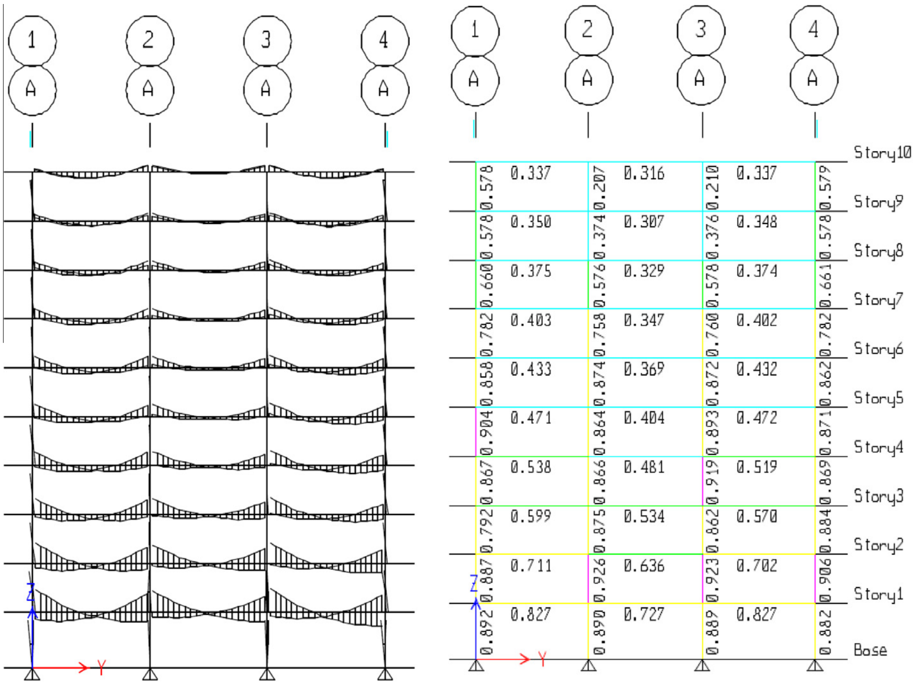

Fig. 4. (a) Bending moment diagram (b) demand/capacity ratio.

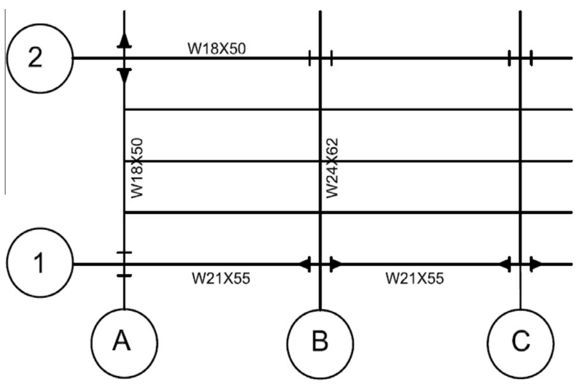

Fig. 5. Partial plan view at location where corner column A1 will be notionally removed at the base level.

Table 2
Calculated m-factors at load-increase area of case study structure.

Beam/girder	Beam/girder m-factor for collapse prevention	Simple connection m-factor for life safety	Moment connection m-factor for life safety
W18 × 50(A1–A2) $\lambda = \frac{b_f}{2t_f} = 6.57 \leqslant \frac{52}{\sqrt{F_{ye}}}$ $\lambda_w = \frac{h}{t_w} = 45.2 \leqslant \frac{418}{\sqrt{F_{ye}}}$	8	–	$2.3 - 0.021d = 1.92$
W24 × 62(B1–B2) $\lambda = \frac{b_f}{2t_f} = 5.97 \leqslant \frac{52}{\sqrt{F_{ye}}}$ $\lambda_w = \frac{h}{t_w} = 50.1 \leqslant \frac{418}{\sqrt{F_{ye}}}$	8	$5.8 - 0.107d_{bg} = 3.26$	–
W18 × 50(A2–B2) $\lambda_f = \frac{b_f}{2t_f} = 6.57 \frac{52}{\sqrt{F_{ye}}}$ $\lambda_w = \frac{h}{t_w} = 45.2 \leqslant \frac{418}{\sqrt{F_{ye}}}$	8	$5.8 - 0.107d_{bg} = 3.874$	–
W21 × 55(A1–B1) $\frac{52}{\sqrt{F_{ye}}} \geqslant \left(\lambda = \frac{b_f}{2t_f} = 7.87\right) \leqslant \frac{65}{\sqrt{F_{ye}}}$ $\lambda_w = \frac{h}{t_w} = 50.0 \leqslant \frac{418}{\sqrt{F_{ye}}}$	6.58 (interpolated)	–	$2.3 - 0.021d = 1.86$

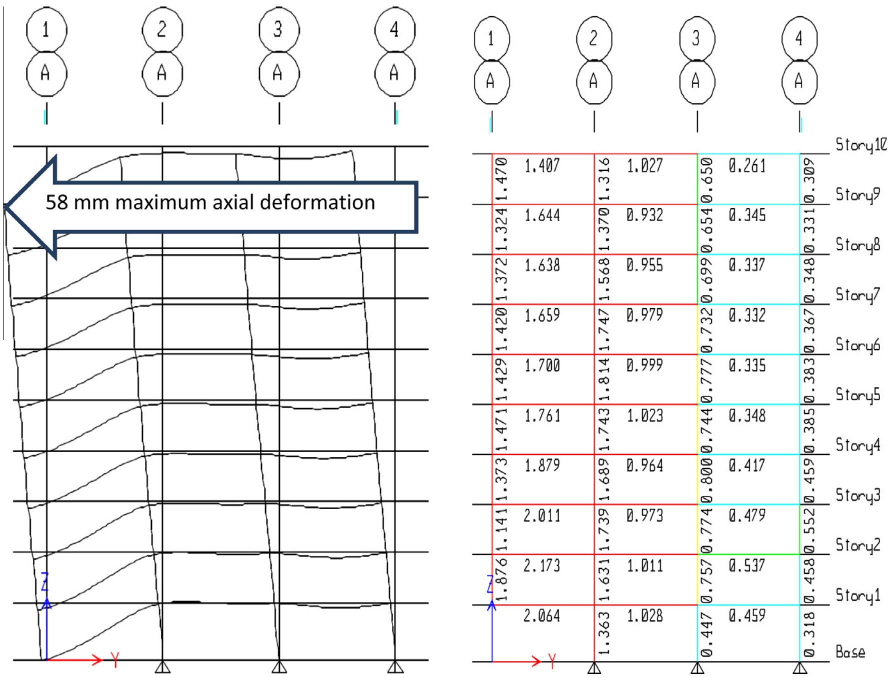

Fig. 6. (a) Deformed shape due to progressive collapse loading. (b) Demand/capacity ratio due to progressive collapse load combination.

Determination of load increase factors for corner column

In this section, the load increase factor for corner column removal is illustrated. Fig. 5 shows partial plan view of the structure at the first floor level below which a corner column will be notionally removed.

The m-factors are needed to determine the load increase factor Ω_{LD} for estimating deformation-controlled actions. It is shown in Table 2 that for different connection type, each beam/girder connected to the notionally removed column is assigned an m-factor for an appropriate performance target, such as *life safety* or *collapse prevention*. If *collapse prevention* is chosen as performance target, significant demand is placed on the structural system. *Collapse prevention* may be chosen as an appropriate performance target for certain earthquake resistant design applications [3]. However, *life safety* is a performance target that cannot be compromised and therefore is chosen in this case study according to the appropriate connection at the end of the beam/girder in the load increase area. The m-factors for the beams/girders in the load increase area of the notionally removed column are determined and summarized in Table 2. The m-factor for the failure of the beam/girder itself is shown in Table 2 for completeness. It is clear that the m-factor associated with the failure of the beam/girder for collapse prevention is large and will also lead to undue demand on the structural system.

Table 2 shows that smallest m-factor of the primary elements directly connected to the notionally removed column is m_{LIF} = 1.86.

Therefore, the load increase factor for the bay immediately above the notionally removed corner column is, Ω_{LD} = 0.9 m_{LIF} + 1.1 = 2.774.

$$G_{LD} = 2.774[1.2\,D + 0.5\,L] = 3.33\,D + 1.387\,L$$

Load on the panels other than those adjacent to the notionally removed column.

$$G_{LD} = 1.2\,D + 0.5\,L$$

The dead and live loads magnified as discussed earlier with G_{LD} were applied to the structure which was then analyzed and designed. Fig. 6a shows the deformed shape due to the progressive collapse load combinations. Significant axial deformation occurred at the location of the notionally removed column at the ground floor level. Fig. 6b shows that the D/C was significantly higher than *unity* in the panels adjacent to the notionally removed column. As expected, the beams A1-A2 failed at all floor levels with significantly higher D/C ratios compared to columns. This is because of higher flexural demand resulting from the 9.0 m cantilevering spans caused by the removal of the corner columns. The beams and columns in the two panels adjacent to the notionally column failed at all floor levels. One panel further from the affected area did not fail.

Summary

- The current approach in progressive collapse resistant design benefited from lessons learned in earthquake resistant structural design. Structural demand and capacity are estimated taking into consideration ductility (or lack thereof) of components.
- Linear and nonlinear static analysis procedures are permitted for a large class of structures. Dynamic effects associated with removal of columns are accounted for through magnified gravity loads. This magnification is applied only in areas tributary to the notionally removed column. The procedure for calculating the magnification factor involves the determination of a factor "m" known as component or element demand modifier, as demonstrated in this paper. The demand modifier originated in the earthquake design research and practice [3].
- Structures with long spans are particularly vulnerable to progressive collapse, especially when corner columns are notionally removed. Stiffening the structural elements may not be practical either. In this case study, challenging spans were used in a typically loaded regular structure. It was shown that the DoD [1] load combinations impose significant demand on the structural system, which may not be able to withstand the loads on its own. Alternate structural solutions, such as outrigger systems [7] may be necessary to resist progressive collapse loads.
- This paper demonstrated the procedure for calculating the magnified gravity loads in areas adjacent to notionally removed columns. It is clear that the number of analyses to capture the entire response is large as the procedure must be applied to several perimeter and interior columns. Furthermore, the procedure must be repeated for each floor for deformation-controlled actions as well as force-controlled actions. This procedure must be automated or otherwise simplified if progressive collapse design based on UFC 4-023-03 [1] is to be adopted by other building codes.

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