Preventing Progressive Collapse in Concrete Buildings

Seismic design details are the key to ductility and load transfer

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Progressive collapse is a chain reaction of failures that propagates throughout a portion of the structure disproportionate to the original local failure. Resistance to it has been primarily a concern limited to rare punching shear failures in flat-plate construction. Its importance, however, has been growing for a wider variety of building types, initially as a response to the Murrah Federal Building bombing and more recently on discussions regarding the events of September 11th. The philosophy of designing to limit damage rather than attempting to eliminate it altogether is similar to the concept adopted in modern earthquake-resistant design.[1]

In general, to prevent progressive collapse, structures should be designed and detailed with an adequate level of continuity, redundancy, and ductility so that alternative load paths can develop following the loss of an individual member. These are characteristics desired in seismic design as well. It should be noted that the current procedures for evaluating progressive collapse are primarily intended to create tougher, more robust buildings, and are in no way an explicit part of blast analysis or design.

For many new and existing construction projects, engineers are now faced with the task of performing progressive collapse analysis that considers the loss of portions of the structure in numerous "missing column" and "missing beam" scenarios. To remain economically viable, this additional design requirement must typically be incorporated without substantial increases in the cost of the structural system. As with addressing natural hazards in the past, engineers must now use their creativity to find cost-effective solutions that will make buildings more resilient to both natural and man-made hazards. One method of achieving this goal is to use a multihazard approach to structural system selection. By choosing systems that can address at the same time the requirements of progressive collapse, seismic, and wind loads, the cost impact can be minimized.

Engineers should select lateral-load resisting systems that can do “double duty,” simultaneously addressing both the lateral and progressive collapse requirements. A building employing interior core walls for lateral-load resistance and ordinary moment frames, or a flat-plate system for gravity loads, for example, may have a very limited ability to redistribute loads and prevent progressive collapse. The main reason is that gravity-load designed systems are not adequately detailed to develop alternative load paths after removal of a primary vertical support (Fig. 1). As illustrated in Fig. 2(a), lack of continuous bottom reinforcement in the beam over the removed column will cause a brittle failure of the resulting two-bay beam. Buildings having special moment-resisting frames (SMRF) for their lateral systems (Fig. 2(b)), however, can provide both the ductility and capacity required to prevent progressive collapse. Findings from the Murrah Building collapse in Oklahoma City, OK, emphasized this point, noting that if special moment-frame detailing had been used, the damage would have been significantly less.[2]

This article illustrates the inherent reserve capacity of seismically detailed reinforced concrete (RC) moment-resisting frames that make the system an excellent first step in designing to prevent progressive collapse. As a demonstration, a study was conducted on a simple 12-story RC frame building representative of existing construction and designed to the older requirements of the Uniform Building Code (UBC) (1991 edition).[3] Knowledge of buildings designed to the older

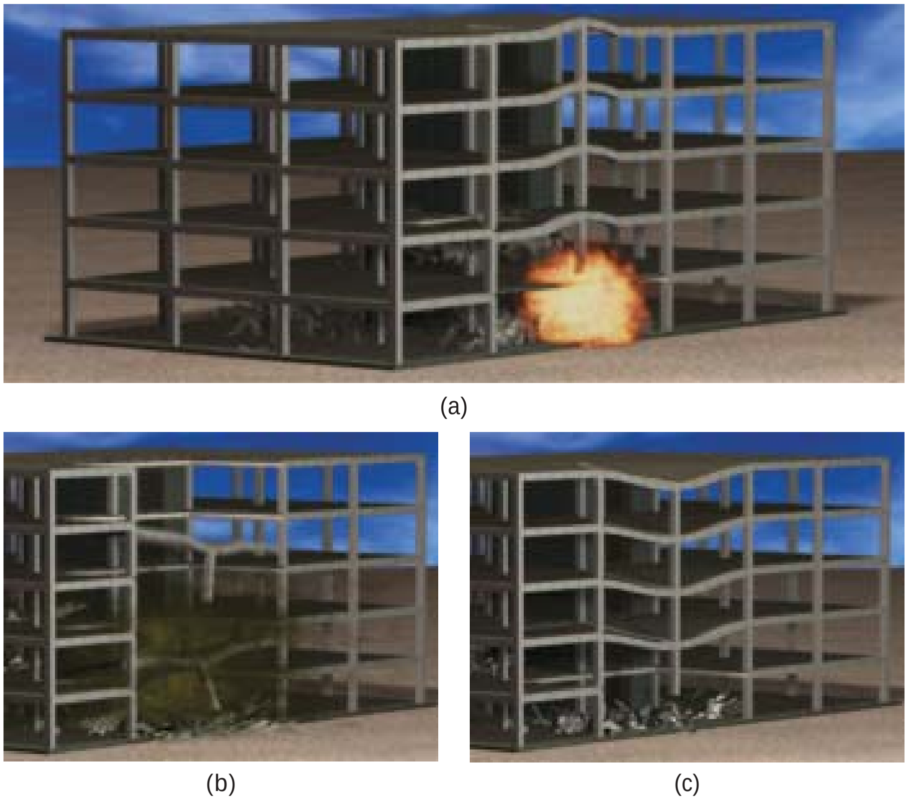


Fig. 1: Effects of losing an external column to a blast loading: (a) exterior blast loading; (b) conventional design: progressive collapse; and (c) alternate load path design: no progressive collapse

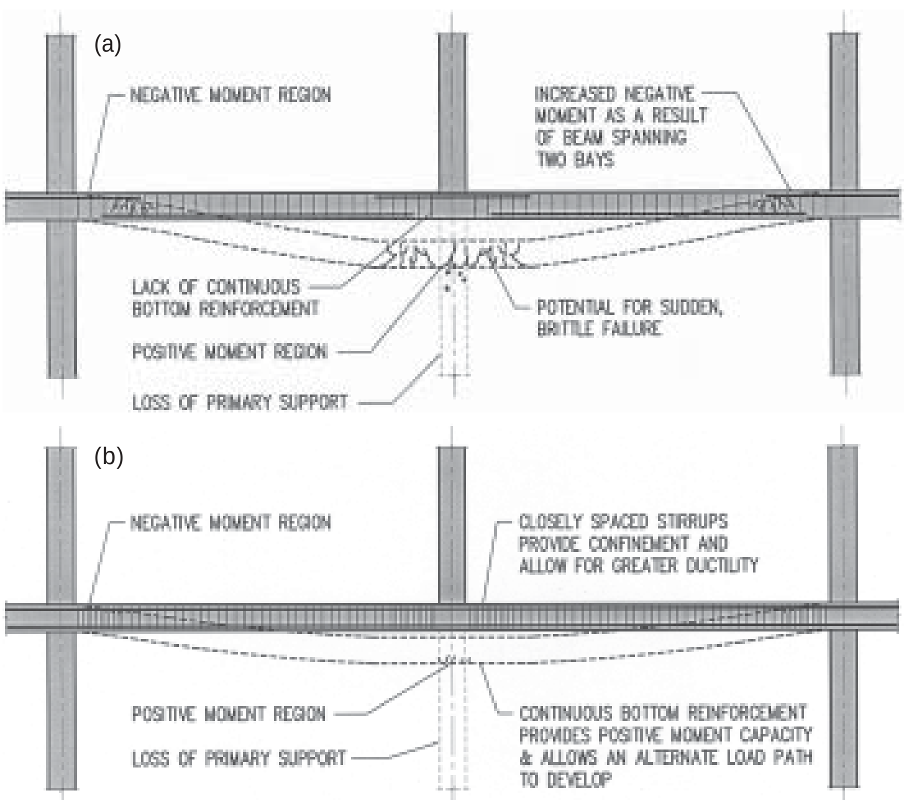


Fig. 2: Response of beam for “missing column” scenario: (a) gravity-load designed beam; and (b) seismically designed beam

TABLE 1:
SECTION DETAILS AND EFFECTIVE STIFFNESS VALUES FOR THE MEMBERS IN THE ETABS MODEL OF THE BUILDING

	Longitudinal beams*		Transverse beams*		Columns		Slabs (two way)	
Zone	2B	4	2B	4	2B	4	2B	4
Size	22 x 18 in.	24 x 20 in.	22 x 20 in.	24 x 26 in.	24 x 24 in.	24 x 24 in.	7 in.	7 in.
Top longitudinal steel	(6) - No. 9	(7) - No. 9	(5) - No. 9	(5) - No. 9	(8) - No. 9	(8) - No. 10	N/A	N/A
Bottom longitudinal steel	(3) - No. 9	(4) - No. 9	(4) - No. 9	(3) - No. 9	(8) - No. 9	(8) - No. 10	N/A	N/A
Stirrups at ends	No. 3 at 4 in.	No. 3 at 4 in. (4 legs)	No. 3 at 4 in.	No. 3 at 5 in. (4 legs)	No. 3 at 8 in.	No. 4 at 3 in.	N/A	N/A
Stirrups at middle	No. 3 at 8 in.	Varies	No. 3 at 8 in.	Varies	No. 3 at 16 in.	No. 3 at 6 in.	N/A	N/A
Flexural ridigity†	$0.5E_cI_g$		$0.5E_cI_g$		$0.7E_cI_g$		$0.25E_cI_g$	
Shear rigidity	$0.4E_cA_w$		$0.4E_cA_w$		$0.4E_cA_w$		$0.4E_cA_w$	
Axial rigidity	E_cA_g		E_cA_g		E_cA_g		E_cA_g	

*Beam dimensions are width x depth.
†I_g based on beam dimensions without effective flange width.
Note: 1 in. = 25.4 mm; No. 3 = D10; No. 9 = D29; No. 10 = D32.

UBC code is valuable for major modernization projects that may require progressive collapse evaluation. For new buildings designed to the more stringent requirements of the 1997 UBC, this analysis will be conservative.

This analysis was performed for UBC Seismic Zone 4 and Zone 2B. The progressive collapse design criteria, as well as the element removal procedure, followed the U.S. General Services Administration's (GSA) "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects."[4] Analysts evaluated each progressive collapse scenario using a 3-D linear elastic model of the structure created in ETABS Plus, Version 7.18 (Extended 3D Analysis of Building Systems), developed by Computers and Structures, Inc., Berkeley, CA.[5]

TABLE 2:
MATERIAL PROPERTIES FOR THE ETABS MODEL OF THE BUILDING

Material	Property	Location	Material strength (ksi): Service condition	Material strength (ksi): Progressive collapse
Concrete	f_c'	Beams and slabs	4	5
		Upper columns	4	5
		Lower columns*	6	7.5
Reinforcing steel	f_y	All members	60	75

*Bottom six stories for Zone 4 and bottom two stories for Zone 2B.
Note: 1 ksi = 6.9 MPa.

MODEL BUILDING

The model building for the study was taken from the design manual entitled *Design of Concrete Buildings for Earthquake and Wind Forces*.[6] The structure consists of five 24 ft (7.3 m) bays in the longitudinal direction and three 24 ft (7.3 m) bays in the transverse direction. Typical floor-to-floor height is 12 ft (3.7 m) except for the first story, which is 15 ft (4.6 m). Service loads of 50 lb/ft^2 (2.4 kPa) live load and 42.5 lb/ft^2 (2.0 kPa) superimposed dead load were assumed in the analysis. The 3-D ETABS model of the

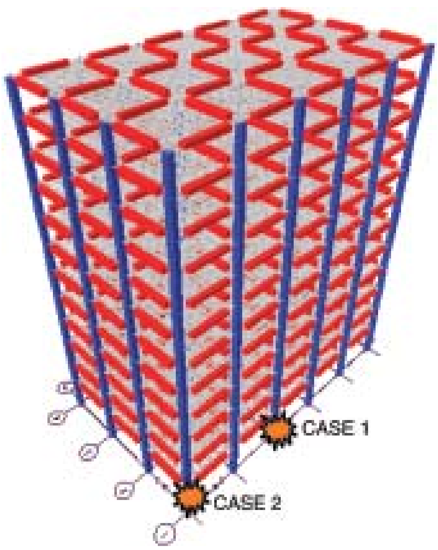


Fig. 3: ETABS model of 12-story reinforced concrete framed building

building is shown in Fig. 3, with pertinent design data summarized in Table 1 and 2.

PROGRESSIVE COLLAPSE GUIDELINES

Currently, there are several standards for evaluating progressive collapse, namely documents by the GSA, Department of Defense,[7] Federal Aviation Administration, and Nuclear Regulatory Commission. While these standards are similar, there are differences. This study investigates the vulnerability of RC moment frames to progressive collapse using the GSA's criteria. It is important to emphasize that this method is prescriptive and does not explicitly consider a specific threat.

The GSA progressive collapse guideline[4] provides a detailed methodology and performance criteria needed to assess the vulnerability of new and existing buildings to progressive collapse. For typical structural configurations, framed structures shall consider the instantaneous loss of a column for one story above grade located near the middle of the long side of the building; near the middle of the short side of the building; and at the corner of the building. A separate analysis must be performed for each case. If underground parking and/or uncontrolled public gathering were to exist under the building, the instantaneous loss of an interior column would also have to be considered.

When performing a static analysis, the vertical load case applied to the structure is as follows:[4]

$$\text{Load} = 2(DL + 0.25LL) \qquad \text{(Eq. 1)}$$

where DL = dead load, and LL = live load.

To avoid an overly conservative design under normal service-load conditions, it is recognized that full live load is unlikely. Thus, in the GSA criteria, live load is reduced to 25% of the full design live load. Multiplying the load combination by a factor of two is the GSA's simplified approach to account for amplification in the response from dynamic effects that can occur when a structural element is violently removed from a structure. In addition, strength increase factors are applied to the properties of construction materials to account for strain rate effects and material over-strength. To determine expected material strengths, the concrete compressive strength f_c' and the yield strength of the reinforcing steel f_y are increased by a factor of 1.25 (Table 2).[4]

Local damage may occur, and is acceptable, with the instantaneous removal of an exterior primary vertical support. The damage, however, must be confined to whichever is smaller: the structural bays directly associated with the instantaneously removed vertical element, or 1800 ft^2 (170 m^2) at the floor level directly above the instantaneously removed vertical member. To evaluate the results of a linear elastic analysis, investigators use the concept of demand-capacity ratio (DCR). It is based on the methodology presented in FEMA-273[8] and FEMA-274[9] for the seismic rehabilitation of buildings. The DCR for structural components is defined as

$$DCR = Q_{UD}/Q_{CE} \qquad \text{(Eq. 2)}$$

where Q_{UD} = demand in component or connection/joint (moment, axial force, and shear) determined from the analysis; and Q_{CE} = expected ultimate, unfactored (ϕ = 1.0) capacity of the component or connection/joint (moment, axial force, and shear). Note ϕ = 0.85 when used for shear.

The concept of a DCR identifies the magnitude and distribution of potential areas of inelastic demand. For a typical structural configuration, an element with a DCR greater than 1.0 has exceeded its ultimate capacity. Failure of an element is imminent if the DCR value for shear (a brittle failure mode) exceeds 1.0 at any section of the element. For continuous elements, however, the flexural DCR value at an element section may exceed 1.0. In this case, flexural demand is then redistributed along the length of the element to sections that have reserve flexural capacity. This redistribution of flexural demand will prevent a flexural failure, provided that:

1. A collapse mechanism has not occurred. A collapse mechanism signifies structural instability caused by hinging at critical sections along a member. Failure of an element will be imminent if a collapse mechanism has occurred; and
2. The element has adequate ductility to redistribute the flexural demand. Based on the

GSA guidelines, structural element sections, or connections that have DCR values that exceed 2.0, are considered severely damaged or collapsed. It is, therefore, unlikely that the structural member or connection will have adequate reserve ductility for effectively redistributing loads.

PROGRESSIVE COLLAPSE ANALYSIS OF MODEL

To more accurately represent the behavior of the structure under different "missing column" scenarios, ETABS[5] was used to generate a three-dimensional model. The selected stiffnesses of the concrete components best represented the stress and strain levels anticipated. Table 1 gives the effective stiffness values recommended in FEMA-273[8] and used in the analysis. E_c represents the modulus of elasticity of concrete; I_g is the moment of inertia of the gross concrete section; A_g is the area of the gross cross section; and A_w is the area of the web cross section. In this study, investigators only considered the removal of exterior elements.

As shown in Fig. 3, Case 1 examined the removal of a column at the first story along the middle of the long side of the building, and Case 2 examined the removal of a corner column, also at the ground level. Because the beams in the transverse direction were designed with greater shear and moment capacity than those in the longitudinal direction, and because the bay sizes are equal in each direction, the removal of a column at the middle of the short side is not a controlling case. For this reason, only the two aforementioned cases were investigated.

This study evaluated only the effect of the progressive collapse analysis on the superstructure of the model building. It is important to note that the capacity of the structure's foundations may also have an impact on overall results. Because the type and size of a building's foundations is largely dependent on local soil conditions, the foundations were not investigated in this analysis. Typically, the results of progressive collapse analysis for the columns should provide a good indication of the adequacy of the foundations.

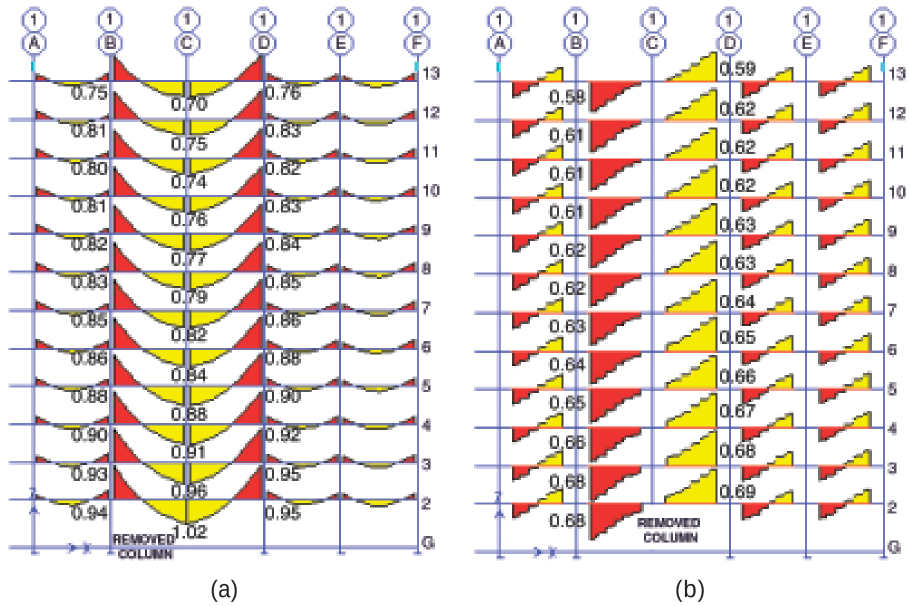


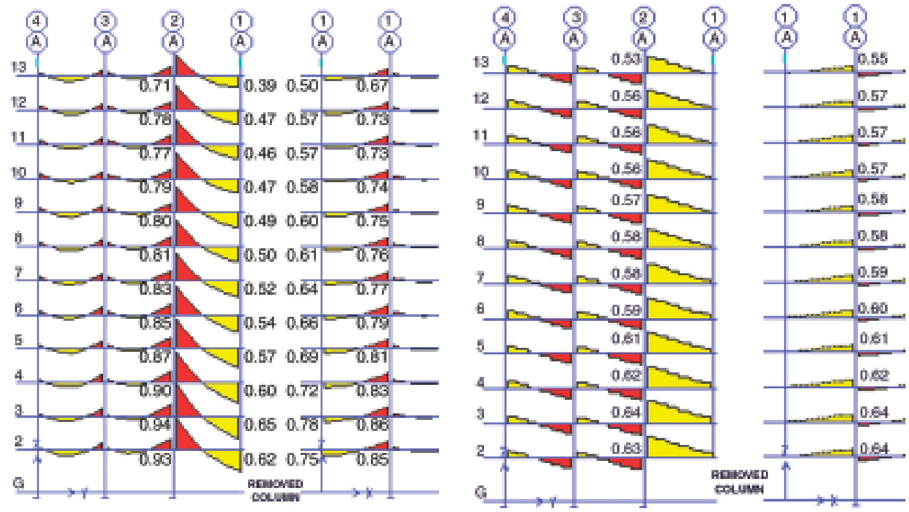
Fig. 4: Analysis results for Case 1–Zone 2B: (a) beam moment diagrams and DCR values; and (b) beam shear diagrams and DCR values

ANALYSIS RESULTS

Zone 2B (intermediate moment-resisting frame)

The removal of a column at the middle of the long side of the building, Case 1, doubles the beam span from 24 ft (7.3 m) to 48 ft (14.6 m). The new 48 ft (14.6 m) beams must be capable of providing an alternate load path into the adjacent columns. As illustrated in Fig. 2, a positive moment is now developed over the removed column. If the beam's bottom reinforcement is not continuous through the column, as in gravity-designed frames, the positive moment capacity is limited to the cracking strength of the section. Failure in this case will be abrupt and potentially catastrophic (Fig. 1(b)). On the other hand, for seismically detailed frames, the use of continuous reinforcing provides positive moment capacity over the removed column and permits the development of an alternate load path (Fig. 1(c)).

As shown in Fig. 4(a), the largest moments for Case 1 concentrate at the first floor and decrease as they move up the height of the building. In addition to presenting the graphical representation of the beam moment diagrams, Fig. 4(a) also shows the DCR values for moment. The DCR values are noted at the midspan (over the removed column) and at both ends of each beam. Calculation of the beam capacities included the contribution of compression steel and strain hardening of the longitudinal steel but did not make allowance for any



(a) (b)

Fig. 5: Analysis results for Case 2–Zone 2B: (a) beam moment diagrams and DCR values; and (b) beam shear diagrams and DCR values

TABLE 3:
SUMMARY OF ANALYSIS RESULTS

Case	Maximum beam DCR–flexure	Maximum beam DCR–shear	Maximum beam deflection	Progressive collapse
1–Zone 2B	1.02	0.69	–4.5 in.	No
2–Zone 2B	0.94	0.64	–4.1 in.	No
1–Zone 4	0.94	0.41	–3.3 in.	No
2–Zone 4	0.86	0.47	–2.9 in.	No

Note: 1 in. = 25.4 mm.

strength increase due to the effective flange width of the floor slab. These capacities closely represent the expected values for the beam. All of the DCR values are below 1.0, except at the midspan of the beam at the first floor. Because this value is only 1.02, however, the beam has enough reserve capacity to redistribute moments to other portions of the structure. Similarly, Fig. 4(b) illustrates distribution of beam shear force and the corresponding DCR values at the beam supports. The maximum DCR value for the beam shear is 0.69, well below the limiting value of 1.0. The maximum deflection, –4.5 in. (110 mm), occurs at the first level directly above the removed column.

Investigators analyzed Case 2 in similar fashion to Case 1. In this scenario, however, all DCR values are below 1.0. Figure 5 presents results illustrating the bending moments, shears, and DCR values in the beams at the corner. To better visualize the moment and shear distribution, the corner is shown as if the exterior face of the building were folded open. It is apparent that a larger percentage of the moment and shear is distributed to the transverse frame. The transverse frame has deeper and hence stiffer beams that attract more of the load. The maximum deflection in this case occurs at the corner of the first floor of the building and is –4.1 in. (100 mm).

Furthermore, for both progressive collapse cases, the DCR values for the columns (under flexure, shear, and axial forces) are well below 1.0. These values signify behavior that should remain well within the elastic range with the removal of an exterior column. In light of the previous results and based on the GSA criteria, progressive collapse is not expected to occur from the removal of a column for either Case 1 or Case 2.

Zone 4 (special moment-resisting frame)

The analysis results for Zone 4 are similar to those for Zone 2B. Unlike the Zone 2B analysis, however, the DCR calculation employed a conservative estimate of the positive and negative moment capacity, which neglected compression steel, strain hardening of the longitudinal steel, and composite flange action. This preliminary estimate was deemed satisfactory because all of the DCR values fell below 1.0 and the structure remained elastic. Maximum deflections for the Zone 4 scenarios occur in the same locations as in the Zone 2B analysis. Table 3 gives the magnitudes of the deflections. As in the Zone 2B analysis, progressive collapse is not expected to occur when an exterior column is removed at the base of the building.

CONCLUSIONS

This study illustrates the inherent ability of seismically designed RC beam-column frames to resist progressive collapse. This knowledge will prove valuable to engineers and architects involved in the selection of structural systems for projects that require progressive collapse mitigation. While the building investigated in this study was rather regular and repetitive in form, the basic analysis procedure can be applied to more complicated structures as well. Based on the results of this GSA evaluation, a number of conclusions can be made:

- For the building configuration considered, the RC moment frames designed for Zone 2B and Zone 4 of the older and less stringent 1991 UBC do not experience progressive collapse when subjected to the "removal" of an external column. New structures designed for the 1997 UBC should actually be more resilient;
- Seismically designed RC moment-resisting frames provide a structure with continuity, redundancy, and ductility. As this study has shown, both new and existing structures designed and detailed with such a system already have an inherent ability to better resist progressive collapse; and
- Through proper structural system selection, progressive collapse mitigation of RC buildings can be enhanced without a substantial increase in project cost.

Author's note

To provide a more in-depth discussion of progressive collapse in concrete structures, the authors are currently working on a PCA publication that should be available at the beginning of 2004.

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