Quantifying and Enhancing the Robustness in Steel Structures: Part 1—Moment-Resisting Frames

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Unusual or abnormal are the adjectives most often used to describe loading imparted to a structural system through events of such low probability of occurrence that they can be omitted in design of the structural system. When a structural system is subjected to an abnormal loading event, there is the probability that a component or components will be compromised and their load-carrying capacity reduced or rendered entirely ineffective. Progressive (disproportionate) collapse describes a scenario whereby localized damage to the structural system propagates into a failure that is much greater than the instigating event. Experience with natural disasters (e.g., earthquakes) and performance of structures in the World Trade Center complex (NIST, 2005) provides evidence that buildings can withstand significant damage and localized component failure without total collapse in the absence of specialized structural engineering consideration for these severe damage states during the original design.

The need to prevent progressive collapse during the design phase and examples of structural systems suffering damage without progressive collapse presents a conundrum to the structural engineering profession. Does the profession stay the previous course under the assumption that nothing needs to be changed? Does it develop load and resistance factor design procedures that include target reliabilities against progressive collapse for all structural systems? Does it recommend that all buildings be designed using such procedures? It is interesting to note that this conundrum is nothing new as the Institution of Structural Engineers faced it in the aftermath of the Ronan Point collapse (ISE, 1969, 1971, 1972a, 1972b).

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The two most commonly referenced United States design guidelines to evaluate progressive collapse resistance in structural systems are published by the General Services Administration (GSA, 2003) and the Department of Defense (DOD, 2005). These guidelines and criteria exercise indirect and direct design procedures (Leyendecker and Ellingwood, 1977). An indirect design approach would be to require that all structural systems have vertical and horizontal ties of minimum specified capacities to facilitate the activation of alternate load paths in the event of an abnormal loading scenario. A direct design approach would be to require the structural engineer to explicitly create alternative load paths in the structural system by considering element removal scenarios and designing members and connections to accommodate the secondary load paths that result. The Unified Facilities Criteria (DOD, 2005) contains a process whereby the structural engineer can align his/her desired protection levels against progressive collapse with a suitable design and analysis procedure. The engineer will implement an indirect or direct design approach based upon the protection level sought. The GSA guidelines (GSA, 2003) essentially guide the structural engineer into a direct design process.

By ensuring orthogonal tying mechanisms are present and/or carrying out element removal with subsequent structural analysis, the structural engineer can enhance a system's ability to resist disproportionate collapse. However, tying a structural system together (e.g., external cabling) may establish conditions whereupon portions of a structural system that is collapsing pull down adjacent portions of the structure (Magnusson, 2004; Loizeaux and Osborn, 2006). These issues add another layer to the conundrum facing the structural engineer. For example, is there a single tying force magnitude that can be used for typical structures? When and where should the ties in the system be broken to facilitate compartmentalization of damage?

ACI 318 (ACI, 2005) has had minimal general structural integrity provisions for decades. Furthermore, other North American specifications for steel design (CISC, 2004) explicitly state that "the requirements of this Standard generally provide a satisfactory level of structural integrity for steel structures." Although one could argue the efficacy of general structural integrity provisions in providing definitive mitigation of disproportionate collapse, and debate the evidentiary support for broad-based statements regarding the levels of structural integrity provided by typical design specifications, there is no denying that provisions and statements such as these give structural engineers and the general public confidence that if a design is undertaken using these codes and specifications, disproportionate collapse is unlikely. General structural integrity provisions and/or general statements regarding the effectiveness of designs carried out using specifications in resisting disproportionate collapse are highly desirable.

The objective of this two-part article is to provide information that can lead to (1) better understanding of disproportionate collapse in structural steel framing systems; (2) improved understanding of secondary load paths that form within structural steel framing systems in the event of a localized failure; (3) development of minimum general structural integrity provisions for structural steel framing systems analogous to those present in ACI 318; (4) recommendations for minimum tie forces that can be used as the basis of indirect design methodologies for structural steel framing systems; (5) an understanding of the distribution of tensile forces within typical steel floor framing systems to facilitate compartmentalization damage or collapse; and (6) identification of simple and economical means with which to enhance the robustness in the typical structural steel framing system.

LITERATURE REVIEW AND SYNTHESIS

Length constraints preclude a detailed literature review. However, there are three sources that the reader can review to gain a complete understanding of the body of literature related to progressive collapse. The first is the annotated bibliography of Leyendecker, Breen, Somes and Swatta (1976). This document covers the body of knowledge accumulated during the time frame 1948 to 1976. Foley, Martin and Schneeman (2007) provide a thorough literature review covering the years 1976 through 2005, and Mohamed (2006) provides a targeted review of state-of-the-art design provisions and several relatively recent research efforts.

The Ronan Point collapse in the United Kingdom was the structural engineering profession's first real-life example of progressive collapse after an abnormal loading event (Griffiths, Pugsley and Saunders, 1968). The report describing the cause of the collapse event and design recommendations to mitigate progressive collapse drew considerable discussion (ISE, 1969, 1971, 1972a, 1972b). The U.K. experience very quickly evolved into building regulations that have been updated and modified through the years (ODPM, 2005).

Following Ronan Point, considerable attention was paid to developing requirements for reinforcement continuity and tie arrangements within concrete structural systems (Popoff, 1975; Speyer, 1976; Breen and Siess, 1979; Fintel and Schultz, 1979). Reinforcement continuity recommendations for slab systems were also developed to mitigate the tendency for disproportionate collapse in the event of a punching shear failure in a two-way slab system (Hawkins and Mitchell, 1979; Mitchell and Cook, 1984). These research efforts serve as the basis for minimum general structural integrity provisions in the current concrete code (ACI, 2005).

Studies quantifying the incidence of abnormal loading events have been undertaken (Leyendecker and Burnett, 1976) and serve as the foundation for probability models for failure (Ellingwood, Leyendecker and Yao, 1983). These efforts eventually migrated into design strategies for mitigating progressive collapse after abnormal loading events (Leyendecker and Ellingwood, 1977; Ellingwood and Leyendecker, 1978; Ellingwood, 2002, 2005; Ellingwood and Dusenberry, 2005; Ellingwood, 2006).

Objective evaluation of codes and design guidelines for progressive collapse mitigative design have been completed (Marchand and Alfawakhiri, 2004; Liu, Davison and Tyas, 2005; Abruzzo, Matta and Panariello, 2006; Ettouney, Smilowitz, Tang and Hapij, 2006; Majanishvili and Agnew, 2006; Ruth, Marchand and Williamson, 2006). There have also been thorough reviews of the progressive collapse mechanism and abnormal loading events with subsequent practical recommendations for mitigating the tendency for disproportionate collapse (Iwankiw and Griffis, 2004; Nair, 2004; Shipe and Carter, 2004; Nair, 2006). Design strategies that can be used to generate structural engineering solutions with low architectural impact have also been proposed (Hamburger and Whittaker, 2004) and the suitability of seismic detailing practice in mitigating progressive collapse in concrete structures has been evaluated (Corley, 2002).

A main focus area of past research efforts has been simulating response of structural systems to abnormal loading events and simulating the progressive collapse mechanism. Girhammar (1980c) examined the role of catenary action in a two-span continuous beam system after loss of interior support and the dynamic loading sensitivity of a small structural system has been evaluated (Christiansson, 1982). McNamara (2003) evaluated the response of a 39-story steel building using static pushdown analysis and an energy-based methodology for evaluating the tendency for progressive collapse following a compromising event has been proposed (Dusenberry and Hamburger, 2005, 2006). Recommendations regarding the use of energy-based procedures for non-single-degree-of-freedom (SDOF) systems and case studies outlining sudden column removal in building systems have been made (Powell, 2005). Rahamian and Moazami (2003) considered a 35-story steel building and several compromising scenarios that involve column removal. Khandelwal and El-Tawil (2005) discuss the results of detailed finite element analysis of large-scale building systems and the tendency for collapsing portions of a structural system to generate large out-of-plane forces on perimeter framing systems. Grierson, Safi, Xu and Liu (2005a) and Grierson, Xu and Liu (2005b) outline several methodologies intended to be used for progressive collapse analysis that include the capability to consider debris loading. General evaluation of two-dimensional steel building systems with partially restrained connections subjected to compromising events has recently been undertaken (Lim and Krauthammer, 2006).

In comparison to analytical work, there have been relatively few experimental efforts undertaken to understand the impact of abnormal loading scenarios on members and connections within a structural steel system. Owens and Moore (1992) evaluated the ability of double angle web cleat and flush end plate connections (considered flexible in the United States) to support tie forces found in British Standards (BSI, 2003). Munoz-Garcia, Davison and Tyas (2005) conducted experimental testing and finite element analysis of these connections to help support and extend this earlier effort through use of high-strain-rate loading protocols. Dynamic testing of a two-span continuous flush end plate connected steel beam with instantaneous loss of interior support was completed (Girhammar, 1980b). Similar testing with extended end plate connections have also occurred (Girhammar, 1980a). Relatively recent testing on simple beam connections (Guravich and Dawe, 2006) provide information on the likely capacities for simple framing connections when subjected to combined shear and tension loading.

A synthesis of the literature leads to the foundation for the present effort. First of all, there are very few studies that provide detailed evaluation of member and connection demands suitable for the development of general structural integrity provisions for steel structural systems. Furthermore, there has been limited study of the impact of framing system topology on the response of steel framing systems to abnormal loading events, and there have been very few efforts undertaken that consider the response of three-dimensional (3D) systems to compromising scenarios. To achieve the objectives listed earlier and address gaps in the body of knowledge revealed through the literature review and synthesis, this two-part article outlines and implements a series of transient and static analyses to quantify demands placed on members, connections, and the floor diaphragm within structural steel moment-resisting framing systems with various topologies when a compromising event occurs. Secondary load paths likely to be activated in the event a structural member is compromised are identified, and attempts to quantify their participation in resisting applied loading are included in a series of three-dimensional inelastic structural analyses. This first paper focuses on moment-resisting frames within the overall building and a companion manuscript; the second paper (Foley, Barnes and Schneeman, 2008) focuses on the typical structural steel floor framing system.

MOMENT-RESISTING FRAMES CONSIDERED

Evaluating the robustness and structural integrity inherent with the steel structural framing system is a relatively daunting task given the fact that there are limitless combinations of shapes, connections and framing configurations. The SAC-FEMA suite of buildings (Gupta and Krawinkler, 1999) were selected as base topologies for the research effort. Pre-Northridge configurations located in Boston were chosen. The framing configurations considered do not satisfy strong-column-weak-beam criteria at all beam-to-column connection locations within the framework. Steel wide-flange shapes with 50-ksi lower-bound yield stress typical of ASTM A992 were used. Framing plans and column schedules for the frames considered are given in Figures 1 through 6. Interior beams and girders were assumed to be connected using *flexible* connections (solid circles at the beam ends). Moment-resisting (fully restrained) connections at the ends of the beams are indicated with solid triangles. A penthouse was located at the roof level and its location is indicated in the plans. Column splice locations are indicated in the column schedules, but these splices were not simulated in the analysis.

The bases of all columns were assumed to be pinned. This is a modification to the three-story SAC-Boston frame. The 10-story building contains one subterranean level and the 20-story frame considers two subterranean levels. Nodes in the analytical model immediately adjacent to the vertical ground edge were assumed to have horizontal movement restrained, but were free to move vertically. The pinned connections at the ends of the infill beams and any girders were considered fully restrained with regard to bending about the member's minor and longitudinal axes (torsion) to reflect the presence of a concrete floor slab. The floor-to-floor heights shown in the column schedules are assumed to be taken from centerline of beam/girder to centerline of beam/girder. No rigid offsets or flexible panel zones were considered. Centerline-to-centerline horizontal dimensions were considered throughout. It should be noted that the structural analysis model included two elements per column member and four elements per beam member. In order to attain a better distribution of mass through the framing system (Powell, 2005), all in-fill beams were divided into two elements.

A relatively simplistic loading scenario was used. The floors were assumed to support a superimposed dead loading of 83 psf, which included concrete-steel composite slab, steel decking, ceilings/flooring/fireproofing, mechanical/electrical/plumbing systems, and partitions (20 psf). The live loading applied to the floors was assumed to be typical office occupancy with a magnitude of 50 psf. The region of the roof in the penthouse area was assumed to have a superimposed dead loading of 96 psf applied. The live loading in this area was taken to be 50 psf. The roof in regions outside of the penthouse area was assumed to support a superimposed

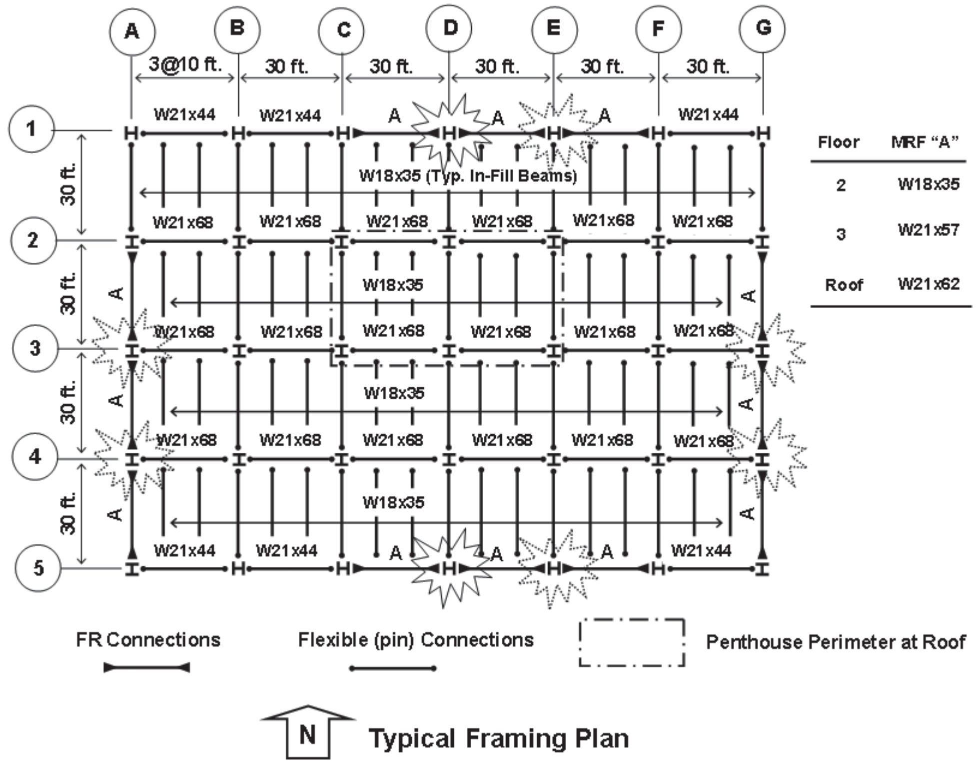


Fig. 1. Framing plan used for the 3-story building frame.

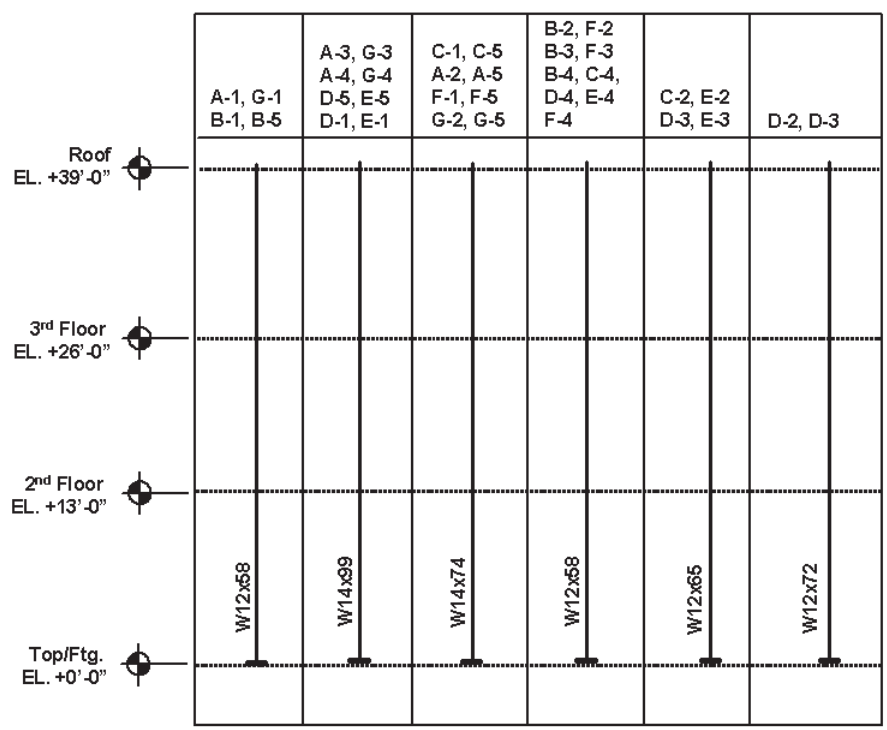
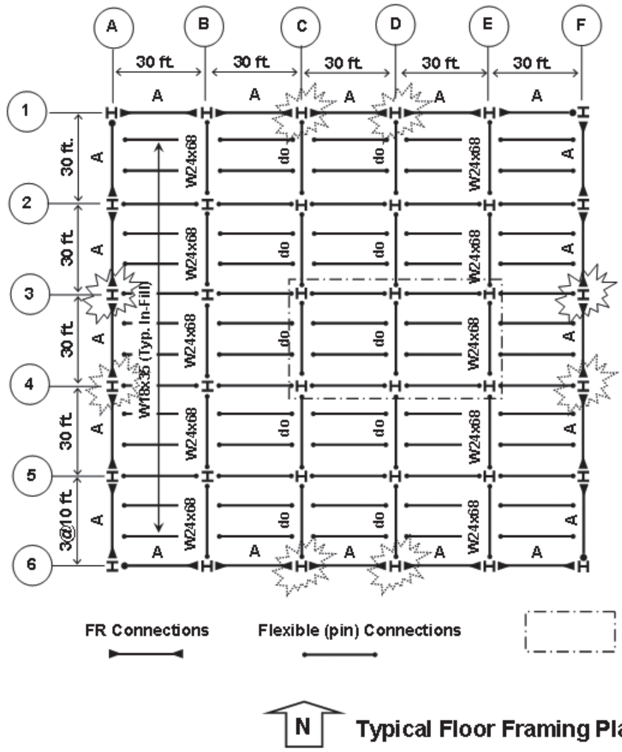


Fig. 2. Column schedule for the 3-story building frame.



Floor	MRF "A"
Ground	W24x76
2	W36x135
3	W33x118
4, 5	W30x116
6	W30x108
7	W30x99
8	W27x94
9, Roof	W24x76

Fig. 3. Framing plan used for the 10-story building frame.

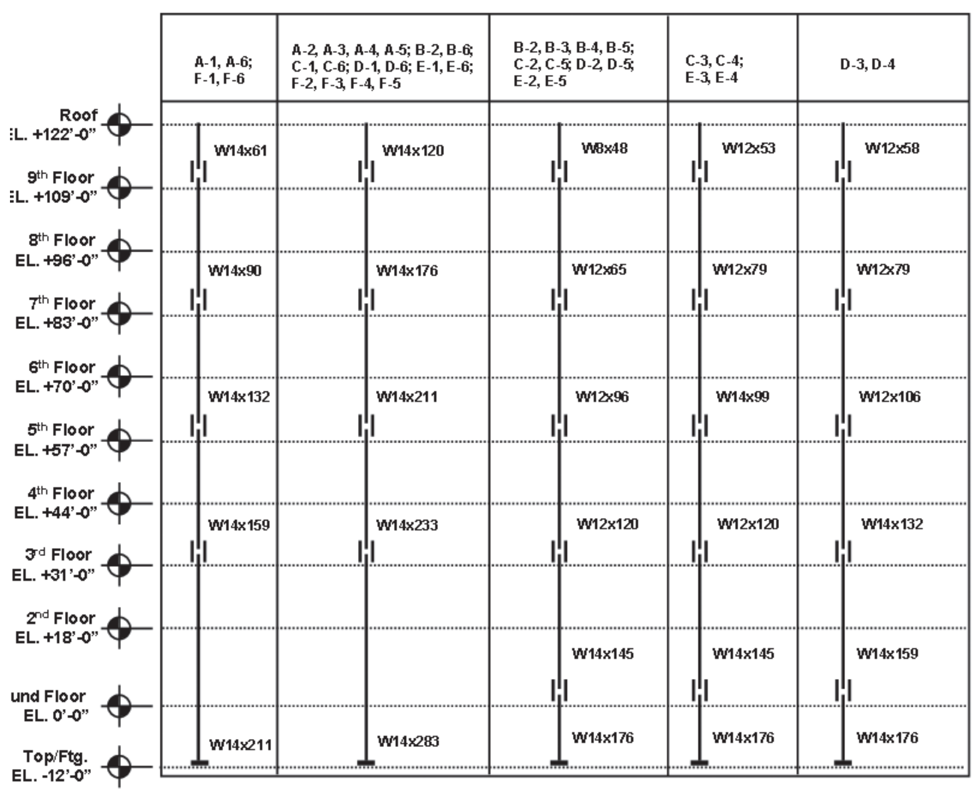


Fig. 4. Column schedule for the 10-story building frame.

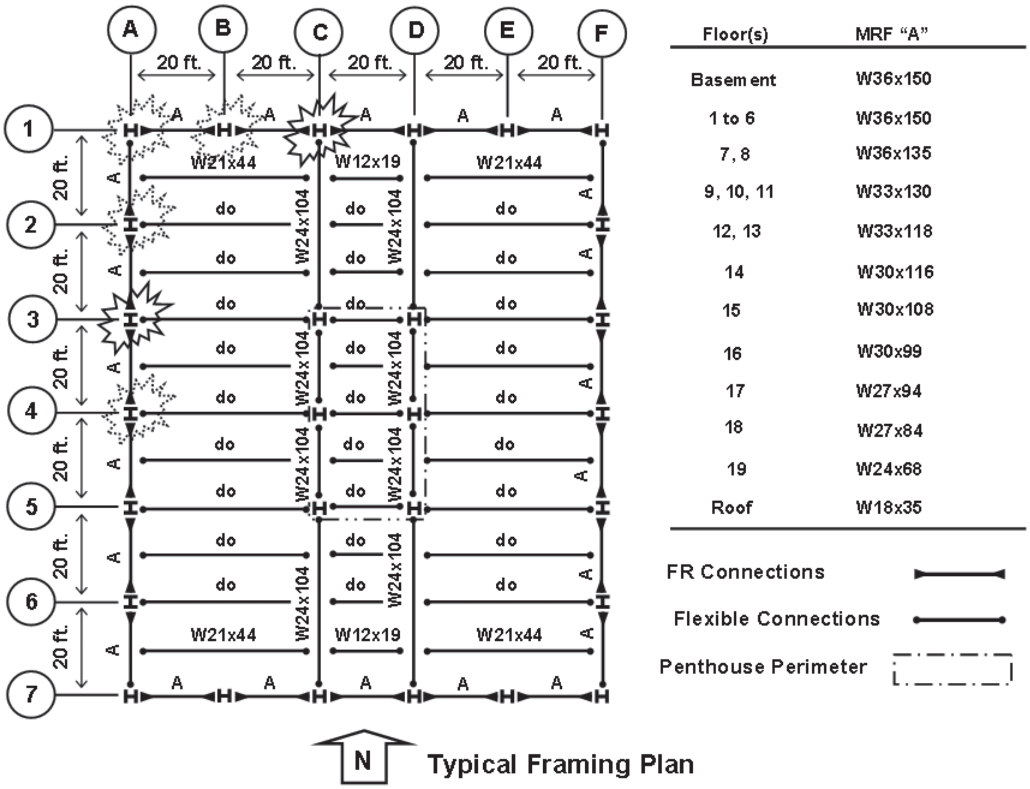


Fig. 5. Framing plan used for the 20-story building frame.

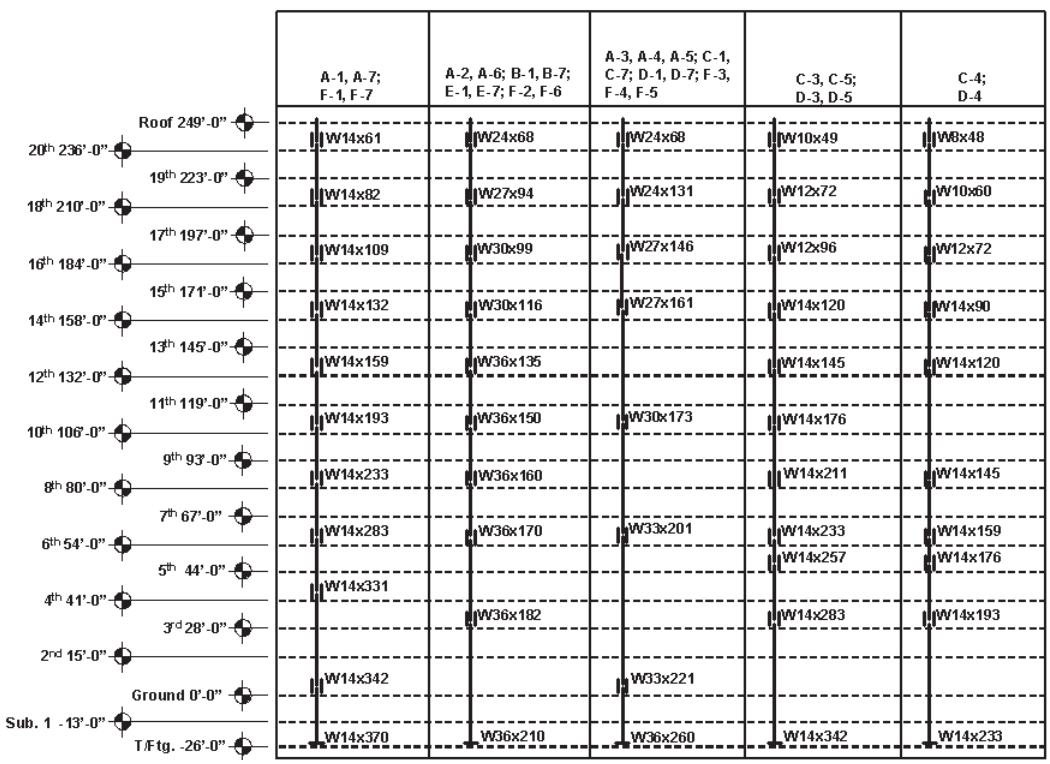


Fig. 6. Column schedule for the 20-story building frame.

dead loading of 63 psf and a live loading of 50 psf (one could argue for a 30-psf snow load). Exterior cladding was assumed around the perimeter of the building. This cladding was assumed to weigh 25 psf over the wall area, including a 3.5-ft-high parapet at the roof level. The self-weight of the structural steel framing members was computed automatically by the computer software.

There are several recommended procedures that can be used to evaluate the tendency for disproportionate collapse (GSA, 2003; DOD, 2005). These procedures were reviewed and the transient analysis approach is used in the present effort. The loading combination applied to the frame at the time of the compromising events is

$$1.0D + 0.25L \tag{1}$$

The live loading used is intended to simulate the live loading present at the time of the compromising event. The live load models used as the basis for U.S. structural engineering involve two components: (1) sustained loading and (2) extraordinary loading. The sustained portion is assumed to be continuously present (with varying magnitude), and it represents ordinary office furniture, bookcases, desks, safes, their contents, and normal personnel (McGuire and Cornell, 1974). Variation in the sustained loading magnitude would likely be generated by tenant occupancy changes. The extraordinary portion of the live loading is intended to simulate those instances where people group during office parties, or cases where office furniture is temporarily stacked during remodeling (McGuire and Cornell, 1974).

The present study assumes that the live loading present when the structural system is compromised is the expected arbitrary point-in-time sustained live loading (Ellingwood and Culver, 1977). Extraordinary live loading components are not considered. Surveys and analysis of office live loading (Culver, 1976; Ellingwood and Culver, 1977) indicate that the expected arbitrary point-in-time sustained live loading is on the order of 11 psf. The present study assumes the live loading present at the time of the compromising event is 12.5 psf. It should be noted that no live load reduction is utilized.

THREE-STORY BUILDING

A three-dimensional structural model was developed for the three-story building using SAP2000 (CSI, 2004). A systematic series of analyses with increasing complexity was conducted. Further details of the modeling and analyses undertaken and results can be found elsewhere (Foley et al., 2007). An initial critical load analysis conducted without diaphragm stiffness modeling resulted in very low elastic critical load factors arising from the lack of a "rigid" diaphragm in the system, and the buckling modes were not consistent with reality. As a result, modeling diaphragm action in the framing system was included. The models implemented in the analysis assumed that the concrete floor slab does not act compositely with the steel skeleton. The slab system deformation that is likely to arise subsequent to a column member within the framing system becoming ineffective is schematically shown in Figure 7. Massless X-bracing members were used to simulate the presence of a composite-steel concrete deck diaphragm at the floor and roof levels through the assumption that the lines of principal tension and compression can be replaced with discrete members with an equivalent shear racking stiffness to that of the concrete portion of the deck system. This concept is very similar to that used by Mahendran and Moor (1999) in the analysis of three-dimensional metal buildings. The present analysis assumed a W14×159 diagonal member with zero material density and eigenvalue analysis of the 3D framework with diaphragm members in place indicated buckling mode shapes that were in line with engineering intuition.

An examination of the framing plan shown in Figure 1 indicates the simply-supported interior framing likely leads to limited ability of the system to overcome an interior or corner column becoming ineffective without activating load resisting mechanisms that are exceedingly difficult to include in the SAP2000 models utilized (e.g., two-way catenary/membrane action in the floor slab). As a result, interior and corner columns becoming ineffective were not considered in this frame analysis. It should be noted that conclusions regarding interior and corner column removal should not be made until two-way catenary action in the floor system,

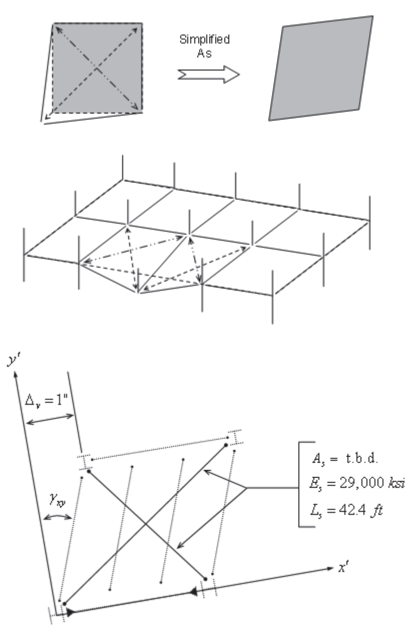


Fig. 7. Conceptualization of diaphragm behavior with a compromised building and diagonal bracing members used to model diaphragm shear deformations.

3D response, and connection characteristics are considered in the analysis. It was decided to examine a scenario whereby an exterior (perimeter) column at the first floor level in the building was rendered entirely ineffective. The ineffective column is located immediately below the second floor level. It should also be noted that only one column at a time is considered to be ineffective. The moment-resisting frame layout in the building considered suggested several compromised column events (highlighted in Figure 1) could be handled with a single analysis. Although the manner in which the gravity load is delivered through the framing system to columns D1, E1, D5 and E5, and A3, A4, G4 and G3 differ, the magnitude of the gravity loading that arrives at each of these columns is the same and a plastic mechanism analysis would illustrate that one group or the other can be considered in the analysis. Therefore, the present analysis considered columns D1, E1, D5 and E5 being compromised.

An elastic buckling analysis was conducted to gain an engineering feel for the system in the compromised state. The first two critical elastic buckling modes were translational sway modes. Higher torsional buckling modes were also present. The magnitudes of the loads at elastic instability were significantly greater than the magnitudes implied in Equation 1 with applied load ratios at instability exceeding 2.0. The elastic critical load analysis was used simply as another check point along the way to detailed nonlinear (geometry and material) transient analysis of the frame.

Elastic dynamic analysis was conducted to evaluate the effect of column loss rates and geometric nonlinearity on the response. The process by which a column in the framework becomes ineffective is modeled using time-history analysis.

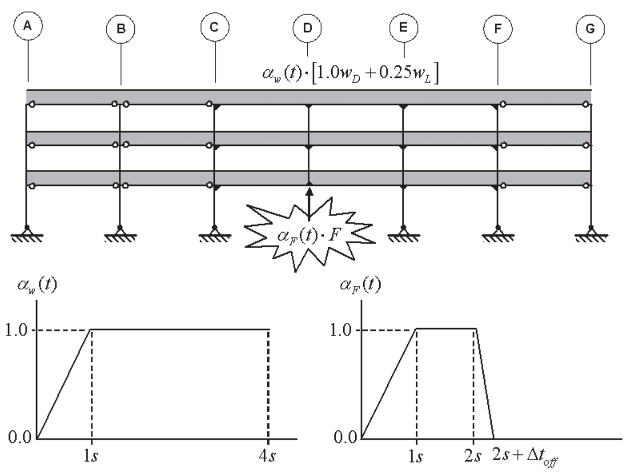


Fig. 8. Conceptualization of column loss scenario implemented in the time-history analysis of the compromised framework.

Both the gravity loading and a concentrated vertical force simulating the loading in the column are applied as time-history functions (Figure 8). The entire analysis duration (4 seconds in this case) was determined via trial and error. The gravity loading and column force are applied over a 1-second interval with an additional 1-second interval with constant magnitude that allows the frame to settle in place with the gravity loads applied. The column's resisting axial load is then turned off over a time interval, Δt_{off}, and the frame is allowed to dynamically respond to this event. One can argue that damping levels during an event whereby beams and/or columns in the framing system become ineffective will be higher than levels during seismic events (i.e., significant deformations, cracking, etc., will occur). However, the present structural analysis assumes damping at a level equal to 5% of critical. Default magnitudes of material-level damping in SAP2000 were also used (CSI, 2004).

The impact of turn-off rate and geometric nonlinearity on the response was examined. Figure 9 illustrates very little difference in peak displacement and the period of the response exists with turn-off rates equal to Δt_{off} = 0.05 s. The response seen with inclusion of geometric nonlinearity is nearly identical to the linear geometric response. This is expected since the applied gravity loading is very small relative to the factored load levels used in design, the axial load in the columns is very small relative to the Euler critical load, and there is minimal translation of column ends relative to one another.

The SAC-FEMA study of moment-resisting connections (FEMA, 2000a, 2000c) pointed out the importance of strain rate. To this end, the elastic strain rates for axial loading, shear loading, and bending moment were computed (Foley et al., 2007). Linear geometric response, 5% damping, elastic material response, and a turn-off rate of 0.01 second were utilized. Local connection effects causing stress concentrations were not considered. The strain rates resulting from axial loading in the columns and beams range from a low of 1,875 με/s (0.0019 in./in./s) and a high of 6,250 με/s (0.0062 in./in./s). The strain rates resulting from transverse shear were significantly higher in all members. Peak shear strain rates occur in the columns of the framework at the top of the third story columns. These rates were on the order of 28,000 με/s (0.028 in./in./s). Strain rates resulting from bending moment for most members in the frame were comparable to the axial and shear rates. An exception is element 1,143, which is a third story column at column line C (see Figure 9). The bending moment normal strain rate for this member is relatively rapid with a peak equal to 50,000 με/s (0.05 in./in./s).

The fracture toughness of steel materials can be reduced as the loading rate increases from that used in determining the fracture toughness according to ASTM E399 (ASTM, 1997; Barsom and Rolfe, 1999). Dynamic and intermediate loading rates are often taken to be on the order of 1.0×10^7 $\mu\varepsilon$/s (10 ε/s) and 1,000 $\mu\varepsilon$/s (0.001 ε/s), respectively (Barsom and Rolfe, 1999). The time to maximum bending moment or maximum shear was on the order of 0.15 second. If it is assumed that the event rendering the column ineffective occurs when the steel's temperature is near room temperature, the loading rates found in the elastic time-history analysis give no indication that fracture toughness of the constituent materials will be significantly diminished. It is understood that local strain concentrations resulting from connection details and flaws that may be generated through welding have been ignored. These issues need further evaluation.

The SAP2000 program has the capability of defining a variety of hinge types in addition to interaction surfaces. Shakedown analysis on benchmark problems led to concerns that the version of SAP2000 (CSI, 2004) being implemented was unable to properly follow axial load moment interaction surfaces; therefore, hinges used in nonlinear material analysis for the present study were limited to moment only, with subsequent checks validating this modeling assumption.

The interaction surface assumed in the present study utilizes nominal moment capacities that include limit states of lateral-torsional buckling and yielding. In those cases where the nominal moment capacity is less than the plastic moment capacity of the cross-section, the moment hinge incorporates a yield moment that is less than the plastic moment capacity. Figure 10 illustrates the moment hinges defined for the inelastic analysis of the three-dimensional framework and their locations within the analytical model. As indicated in the figure, the moment capacities of the W14×74 and W14×99 cross-sections are limited by flange local buckling. The moment hinges were defined using the expected yield stress of the material. Points B, C and D are defined with a small amount of hardening to aid in convergence during the nonlinear solution.

Inelastic time-history analysis was carried out by starting with a model that was a modification of the elastic analysis model. Major-axis plastic-moment hinge behavior was modeled at the locations indicated in Figure 10, and the time-step increments used during the Newmark solution algorithm were reduced to 0.001 second. The analytical hinges indicated in the figure were friction-free pins inserted in the model to conservatively simulate flexible connection behavior at the indicated locations. The convergence tolerance was relaxed slightly from the default values, and the event tolerance parameter was set to 0.01. The same time-history functions and column upward axial load magnitudes were used in the inelastic analysis. Because the elastic time-history analysis demonstrated that nonlinear geometric effects were negligible, a first-order materially nonlinear analysis was executed here. The system deformations seen in the inelastic analysis support this assumption.

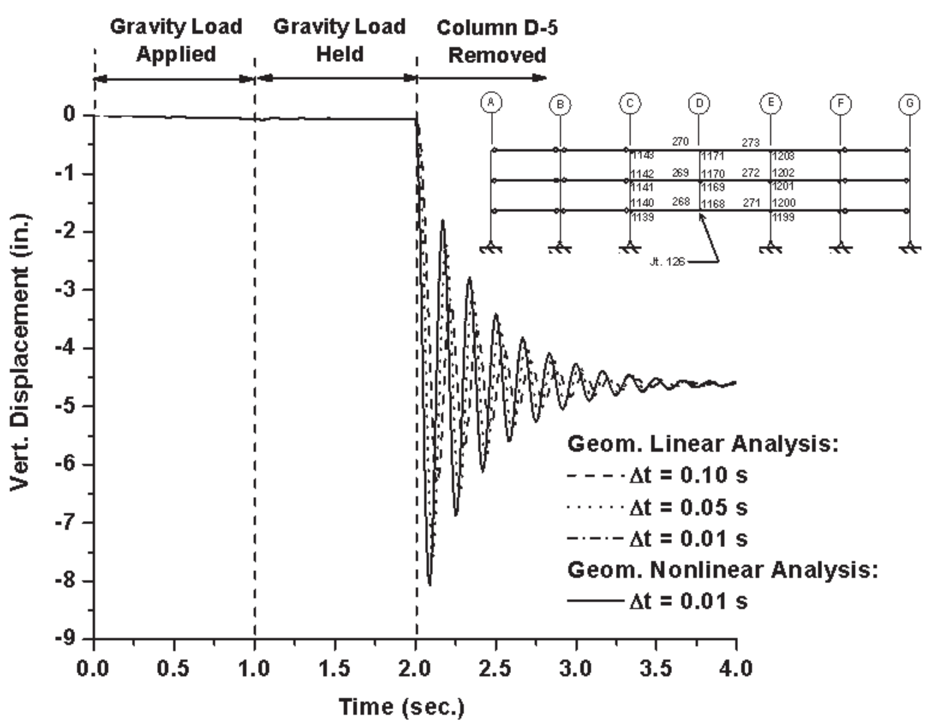
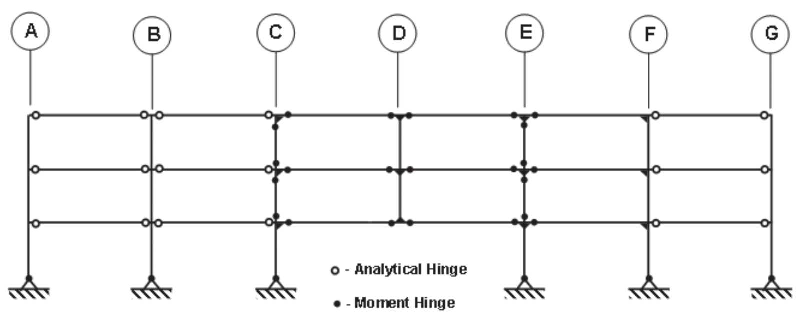


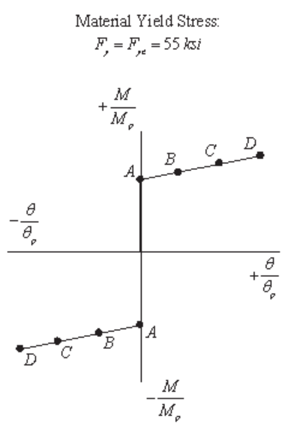
Fig. 9. Impact of column turn-off rates (Δt) on elastic linear and nonlinear geometric response of the 3-story framework.

Figure 11 illustrates the peak inelastic displacement is 50% greater than the elastic displacement. This 12.5-in. peak displacement is essentially 1.7% of the 60-ft span length in the compromised system. A flexural hinge collapse mechanism did not form in the framework at this imposed demand, and the use of geometrically linear analysis is justified (a later figure illustrates the hinge mechanism). Furthermore, assumptions regarding catenary action as a primary load transfer mechanism preventing progressive collapse should be carefully evaluated. This confirms earlier recommendations (Powell, 2005). Catenary forces can become substantial if elastic flexural behavior is demanded. The structural engineer should seek balance between primary flexural resistance and secondary catenary action. The frame's configuration after the event resulted in 11 in. of permanent deformation. Plastic hinging is present in this configuration, but a flexural collapse mechanism has not formed. Loading combinations used for static analysis recommended often include a 2.0 multiplier to simulate dynamic loading effects (GSA, 2003). The results for the elastic and inelastic analysis in Figure 11 appear to indicate that this is conservative as suggested by others (Marchand and Alfawakhiri, 2004).

The X-bracing diaphragm simulation used in the analysis affords an opportunity to examine the activation of the floor slab system as a column is compromised. Figure 12 shows the three floors in the framing system and the axial forces present at the instant the peak vertical displacement is reached in the time-history analysis. The diaphragm system is activated from front to back of the structural system, and the floor slab should not be ignored when assessing general structural integrity. Bending stiffness in the floor slab system is ignored in the present analysis, and it is likely that this is important at the vertical deformation magnitudes seen in the analysis. It is also important to note that the axial forces in the girders on either side of column line D are preserved. Rigid diaphragm modeling would result in these axial forces being zero. The axial tension magnitudes present in the diagonal members at the second and third floors indicates that the system may be amendable to damage compartmentalization within the influence area of the column line containing the ineffective column at these levels. The roof level contains significant compression forces.

The bending moment response history for three members coming together at the second floor at column line C is shown in Figure 13. At shortly after 2 seconds, a plastic hinge forms in beam 268 (refer to Figure 9 for element indexing). This hinge serves to cap the bending moment at the corresponding beam-to-column joint. As a result, the





Section	Point	$\frac{M}{M_p}$	$\frac{\theta}{\theta_p}$
W18 W21	A	1.00	0
	B	1.05	8
	C	1.10	16
	D	1.15	24
W14×74	A	0.88	0
	B	0.94	8
	C	0.99	16
	D	1.04	24
W14×99	A	0.99	0
	B	1.04	8
	C	1.09	16
	D	1.14	24

Fig. 10. Moment hinge locations and modeling parameters used.

bending moments in adjacent columns are capped as well. Thus, the hinging in beam 268 serves to protect the columns framing into the common joint. Bending moment response histories for members 270 and 1,143 are shown in Figure 14. In this case, the column forms the plastic hinge first, and the demand in the adjacent beam is immediately capped at that moment magnitude. It is interesting to note that the elastic and inelastic response dampen out to the same static bending moment capacity as required by equilibrium and yielding is temporary. The bending moment response histories at each end of element 268 given in Figure 15 illustrate complete reversal of bending moment as the column is rendered ineffective. The hinge formations that occurred during the structural analysis are graphically depicted in Figure 16.

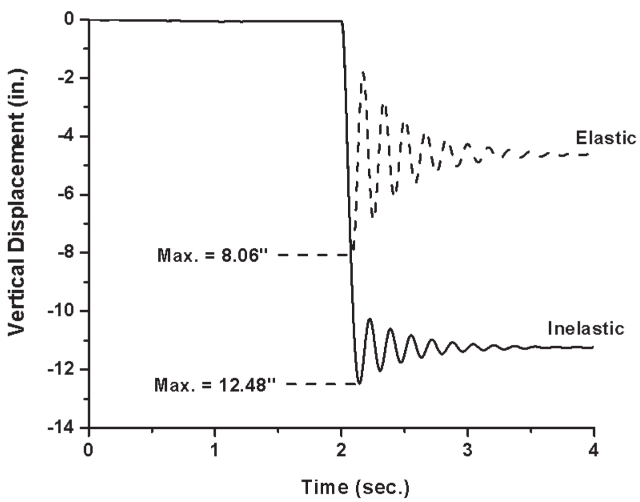


Fig. 11. Response comparison at joint 126 for three-story frame with ineffective column at first floor and location D-5.

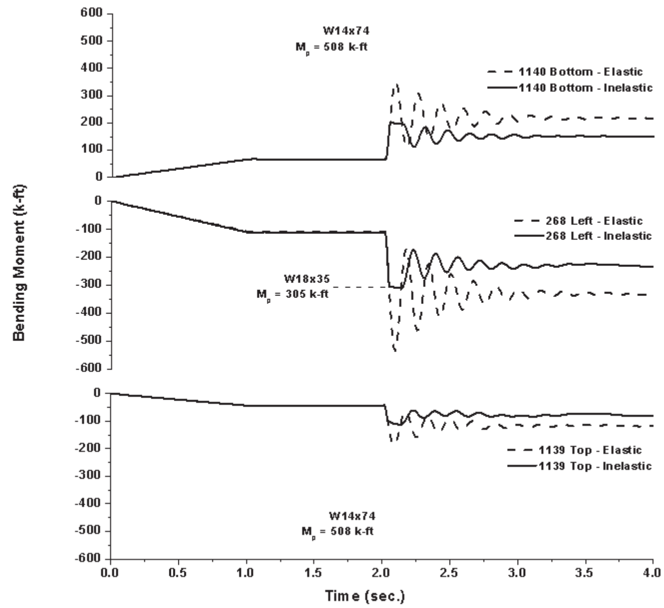


Fig. 13. Bending moment time-history response for members framing together at the second floor beam-to-column connection at column line C.

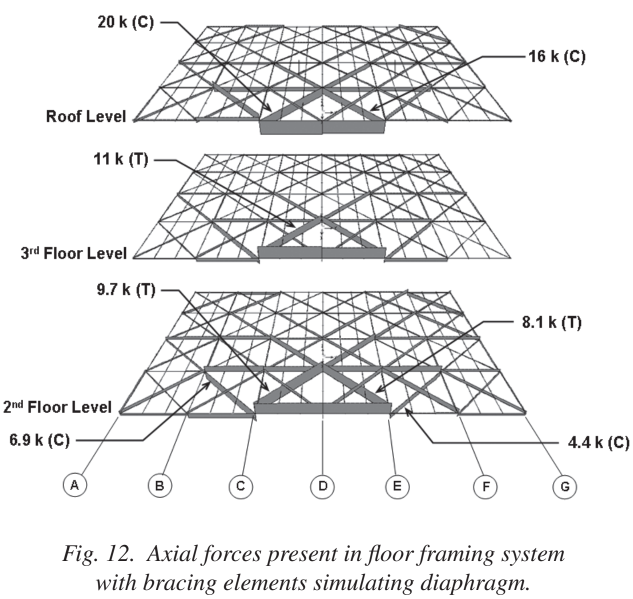


Fig. 12. Axial forces present in floor framing system with bracing elements simulating diaphragm.

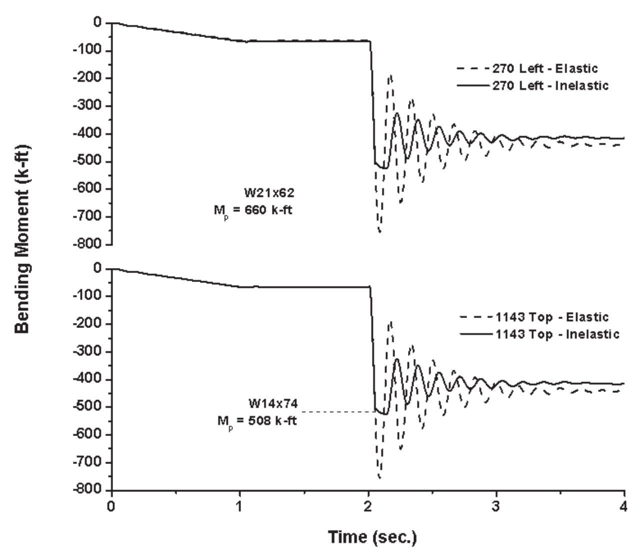


Fig. 14. Bending moment time-history response for members framing together at the roof beam-to-column connection at column line C.

In general, the peak transient bending moments seen in the members are much lower when inelastic behavior is included (refer to Figures 13 through 15). The response seen also stresses the importance of designing moment-resisting connections for full reversal of moment and robustness in the structural steel framing system can be enhanced by designing moment-resisting connections for equal moment magnitudes in positive and negative bending.

Because the moment hinges utilized in the structural analysis did not consider axial-load bending-moment interaction, a demand-to-capacity ratio was utilized to help quantify the extent to which the cross-section yield surface may or may not be violated. The demand-to-capacity ratio (DCR) is defined as follows,

$$\mathrm{DCR} = \frac{P}{P_n} + \frac{M}{M_n} + \left(\frac{V}{V_n}\right)^2 \tag{2}$$

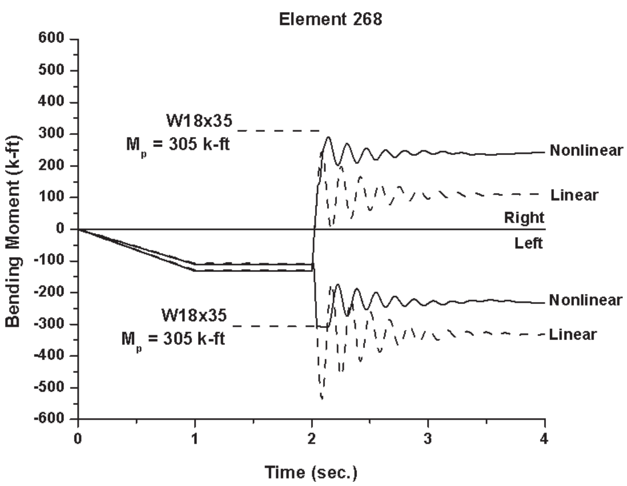


Fig. 15. Bending moment time-history response for second floor beam (element 268) spanning from column line C to D (nonlinear indicates material nonlinearity considered).

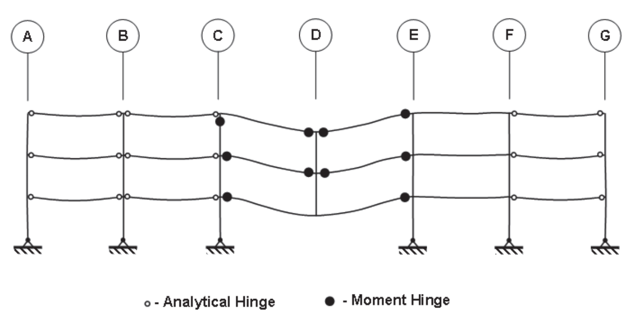


Fig. 16. Moment hinge formation computed during response of frame to ineffective column at the first floor at location D-5.

It should be noted that the DCR defined in this manner is different than that used in seismic specifications (FEMA, 2000b) and progressive collapse mitigative design guidelines (GSA, 2003).

The demand-to-capacity ratio expressed in Equation 2 is conservative with regard to the interaction of bending moment and axial loading. Traditional interaction equations for wide-flange shapes (AISC, 2005) do not involve the quadratic term accounting for the influence of shear. However, the interaction equations for HSS do involve a quadratic shear contribution (AISC, 2005). Quadratic (circular) models for the interaction of transverse shear and bending moment have a long history (Hodge, 1959). Rectangular yield surfaces (no influence of transverse shear on the plastic moment capacity) for modeling the interaction of shear and bending have also been utilized in eccentrically braced frame (EBF) research (Ricles and Popov, 1994).

The AISC *Specification for Structural Steel Buildings* (AISC, 2005), hereafter referred to as the AISC *Specification*, is utilized to determine the nominal axial capacities, shear capacities, and bending moment capacities for the members. The nominal axial capacities of the vertical elements in the framework are defined using an effective length factor for both axes of flexural buckling equal to 1.0 and a C_m factor of 1.0. Moment amplification for P-δ and P-Δ effects were ignored because the axial loading in the members is much less than the Euler critical load and translation of the column ends relative to one another was negligible. The unbraced length of the member was taken to be the story height (13 ft). Horizontal element (i.e., beams) axial strengths were taken to be full yield strengths using expected yield stress with the assumption that connections are adequate. In compression, the concrete slab is likely to take a significant amount of compression force, thus limiting axial compression carried by the beam. The bending moment capacity for the vertical elements in the system are defined using pure-bending modifiers of C_b = 1.0, which is conservative. Horizontal element bending capacity is determined assuming the fully braced condition. Local buckling was considered in the definition of bending capacity for all elements in the system. The shear strength of the member is defined using the height of the web in the cross-section (depth minus two flange thicknesses) and the web thickness. The shear capacity of the web is taken from the AISC *Specification* (AISC, 2005).

Time-histories of nondimensional axial load, shear, and bending moment demands along with DCRs for member 1,143 are given in Figure 17. The majority of the DCR is contributed by the bending moment demand and the axial load demand throughout the time-history is low (e.g., less than 10% of nominal capacity). The DCR for member 1,143 nears 1.25, which exceeds 1.00 by a significant margin. If the load and resistance factor design method were utilized to evaluate the capacity of this member, the nondimensional

Table 1. Peak Nondimensional Member Demands and Demand-to-Capacity Ratios for Inelastic Response to Ineffective Column at Location D-5

Member (1)		$\frac{P}{P_n}$ (2)	$\frac{V}{V_n}$		$\frac{M}{M_n}$		$DCR = \frac{P}{P_n} + \left[\left(\frac{V}{V_n}\right)^2 + \frac{M}{M_n}\right]$	
			Left/Bot. (3)	Right/Top (4)	Left/Bot. (5)	Right/Top (6)	Left/Bot. (7)	Right/Top (8)
BEAMS	268	0.018	0.255	0.126	1.014	0.950	1.095	0.967
	271	0.016	0.121	0.256	0.959	1.015	0.976	1.097
	269	0.019	0.244	0.082	0.997	0.990	1.071	1.012
	272	0.017	0.077	0.245	0.990	0.998	1.011	1.071
	270	0.037	0.231	0.080	0.796	0.994	0.882	1.037
	273	0.034	0.103	0.251	0.997	1.000	1.039	1.092
COLUMNS	1139	0.336	—	0.046	—	0.223	—	0.558
	1140	0.240	0.182	—	0.403	—	0.665	—
	1141	0.240	—	0.181	—	0.470	—	0.605
	1142	0.113	0.361	—	0.701	—	0.942	—
	1143	0.113	—	0.361	—	1.038	—	1.278
	1168	0.133	—	—	—	—	0.133	—
	1170	0.038	—	—	—	—	0.038	—
	1199	0.227	—	0.020	—	0.067	—	0.285
	1200	0.166	0.102	—	0.158	—	0.328	—
	1201	0.166	—	0.101	—	0.196	—	0.358
	1202	0.083	0.253	—	0.347	—	0.491	—
	1203	0.083	—	0.253	—	0.503	—	0.647

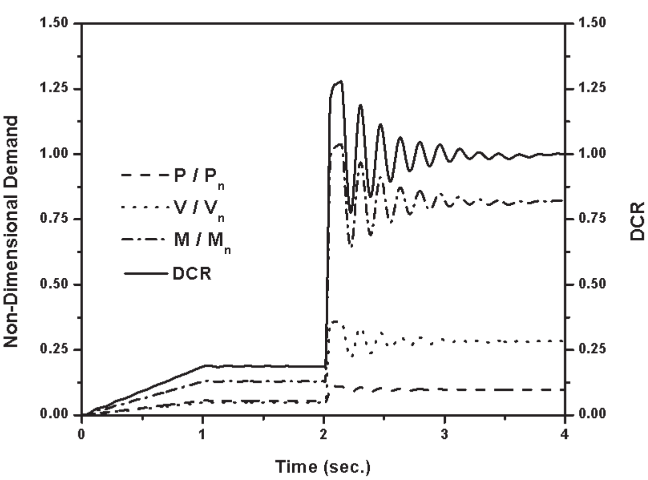


Fig. 17. Nondimensional force demands and demand-to-capacity ratio time-histories for element 1143 at the top (roof level).

axial demand component would be reduced by a factor of 2, and as a result, the DCR would migrate toward 1.00. Furthermore, the analytical model assumed that a friction-free pin connection exists at the left side of column 1,143 (see Figure 9). In all likelihood, this connection will not be a friction-free pin, but would have a moment capacity on the order of 10% of the plastic moment capacity of the connected beam. This would further reduce the demand on the column and the DCR would therefore reduce further.

The peak nondimensional demands and DCRs seen in the simulation conducted are given in Table 1. The peak axial load demands for the beam members are all low (e.g., less than 4% of the axial tension or compression capacity of the member's cross-section). The peak axial force demand for member 1,143 is seen to be 11%, and the bending moment demand is slightly greater than 1.00. The peak tension forces occur in the floor beams immediately above those adjacent to the compromised floor level. The maximum tension force is 17.5 kips, or approximately 2% of the tension capacity of the girder (Foley et al., 2007). It should be noted that compression force will likely be carried by the floor slab and beam in a symbiotic manner.

One additional item of importance in the assessment of the frame's ability to compensate for an ineffective column and prevent disproportionate collapse is the plastic rotation demands that are placed on the connections at the ends of the beams. Plastic rotation was computed using the displacement results taken from the inelastic time-history analysis. The displacement and moment demand data indicated plastic rotation demands that were fairly consistent across all members. The plastic rotation demands were on the order of 0.022 rad in the girders. If the plastic rotation at the top of member 1,143 is assumed to provide all rotation at the joint facilitating vertical movement at column line D, then the plastic rotation in this member is on the order of 0.016 rad.

The plastic rotation demands encountered during the compromising event are significant. A recent compendium of connection testing information (FEMA, 2000c), indicated a wide variety of plastic rotation capacities for typical fully restrained moment-resisting connections with many exceeding the 0.022 rad demand predicted. Connections that give rise to concerns are welded-flange-bolted-web connections that utilize E70T-4 electrodes without minimum notch toughness and higher yield-to-tensile stress ratios and welded-flange-bolted-web connections utilizing E70T-4 electrodes with the backing bar removed. Welded-flange-bolted-web connections with improved access holes, free-flange connections, cover-plated connections, RBS connections, and welded flange plate connections all were shown to have expected (mean) plastic rotation capacities that exceed the peak demand computed. It is expected that a plastic hinge forming in the W14×74 member can sustain 0.016 rad of plastic rotation prior to significant moment capacity degradation. In many cases, the time-history analysis indicated that the plastic capacity of the cross-section was temporarily reached (refer to Figure 15) and inelastic reloading did not occur.

10-STORY BUILDING

A 10-story frame was also chosen for detailed structural analysis for compromising events. This frame was described earlier in Figure 3 and a column schedule describing the members was given in Figure 4. It is expected that the tension and compression demands on the connections in this framework will vary with the ability for deep-beam action to form within the structural system. Thus, the 10-story frame is considered to examine the variation in demand placed on the members and connections when additional framing is present above the compromised area. An overview of the analysis conducted on this frame is given in this paper, and details are given elsewhere (Foley et al., 2007).

Compromised interior and corner columns were not considered for the same reasons cited earlier with the 3-story frame. The three-story frame loading and combination given by Equation 1 was used. The framing plan suggested evaluation of the compromised column events shown in Figure 3. Slab membrane effects and the response characteristics for the connections at the ends of the orthogonal in-fill beams and the girders are likely to be very important but were not considered in the analysis. Zero-density X-bracing members were utilized to simulate the presence of a concrete and steel deck diaphragm system. The time-history function used to simulate column loss utilized a column turn off rate equal to 0.01 second.

The impact of geometric nonlinearity (P-δ or P-Δ effects) on the response was evaluated using a column turn off rate of 0.01 second. The peak vertical displacements in the floor immediately above the ineffective column were on the order of 2.5 in., indicating geometric stiffening (i.e., catenary action) is not present. Column ends did not tend to translate with respect to one another, and the axial loads in the columns were well below the Euler critical load; therefore, loss of stiffness due to geometric effects is negligible.

The peak strain rate caused by axial loading in the columns and beams was 5,650 με/s (0.0057 in./in./s). The largest strain rates caused by transverse shear occurred in the columns, and these rates were on the order of 105 με/s (0.00011 in./in./s). Bending strain rates in the columns were comparable to the strain rates caused by axial loading with a maximum of 2,000 με/s (0.002 in./in./s). As seen in the three-story building analysis, the strain rates seen in this structure are only slightly more than the intermediate loading rate and orders of magnitude lower than the dynamic loading rate (Barsom and Rolfe, 1999). The loading rates found in the elastic time-history analysis give no indication that fracture toughness of the constituent materials will be insufficient or that the elevated yield strength resulting from increased load rate should be considered. It should be emphasized again that stress raisers caused by connection geometry have been ignored.

The need to determine if inelastic time-history analysis was warranted was performed by evaluating demand-to-capacity ratios (DCRs) for the members in the framework as defined previously in Equation 2. The elastic time-history analysis indicated that the largest DCRs are found in the beams, which had a peak value of 0.68, and the columns on either side of the column rendered ineffective had DCRs less than 0.63 (Foley et al., 2007). Overall, the peak DCRs for the critical members in the vicinity of the ineffective column indicated that inelastic response is unlikely and no plastic hinges formed in the framing members in this framework. Peak elastic tie force demands in this system were determined to be $0.02P_n$, which is very similar to the demand seen in the three-story frame where the response was inelastic.

In the case of the W36×135 girders used in the present framework, this works out to be 0.02(50 ksi)(39.7 in.^{2}) = 40 kips.

As the number of stories above the compromised column increases, a deep-beam mechanism exhibited by beam tension and compression forces is activated. The axial load distribution seen in the beams above the compromised column is analogous to the normal stress (strain) distribution seen in beams with very deep cross-sections. The axial loads seen in the beam members include many levels of compression being equilibrated with a single relatively large tension force in the beams immediately above the compromised column. As a result, the axial tension loading is much greater in the beams/girders of this frame when compared to the three-story building (17.5 kips versus 40 kips). Furthermore, the peak tension forces in the three-story frame occurred simultaneously with the primary flexural hinging mechanism. Because the 10-story frame responds in an elastic manner, this flexural hinging is not present.

20-STORY FRAME

A 20-story structure based upon the SAC-FEMA project documents (Gupta and Krawinkler, 1999) was also analyzed. The pre-Northridge Boston design was used as the base topology, and the framing plan for this building and its column schedule were given in Figures 5 and 6. Load magnitudes were the same as those used for the three- and 10-story buildings. A slightly modified version of the X-bracing diaphragm modeling was used for this framework. Elastic critical load analysis of the framework demonstrated that the massless X-bracing members were sufficient to model diaphragm action (Foley, Barnes and Schneeman, 2008). Several minor changes were made to the frame found in the SAC document (Gupta and Krawinkler, 1999). These changes were focused on the sizes of the infill beams and several column members in the interior of the framework. The nodes in the analytical model for the building are restrained from lateral movement at the lowest two subterranean floor plates, but vertical movement and rotation was allowed. Translational restraints were provided at the bottom of each column. Details of the modeling and analysis completed for framework can be found elsewhere (Foley et al., 2008).

Elastic time-history analysis conducted for the 20-story frame considered a variety of column removal scenarios at the ground-floor level. Because the perimeter framing contained moment resisting frames where all girders and beams were rigidly connected to the columns (with the exception of the corner columns in the east and west frame elevations), six column-removal scenarios were identified for analysis (Figure 5). Each involved rendering one column at the ground floor level ineffective. The columns were chosen based on symmetry of the floor plan and their ability to represent generalized response.

Preliminary time-history analysis indicated that linear geometric analysis was sufficient to assess response of the frames if elastic material response occurred. The initial analyses conducted assumed linear material response and the validity of this assumption was subsequently evaluated through use of demand-to-capacity ratios for members. The removal of columns A3 and C1 were found to produce the most significant results and these scenarios were selected for detailed analysis. The framing plan used in this structural system results in very low levels of axial loading in the corner columns and therefore, loss of column A1 was not critical. Maximum vertical displacements at the floor level immediately above the ineffective column occurred between 0.0 and 0.25 second after the column was rendered ineffective. The peak vertical displacement for these two scenarios was 0.89 in. and 0.81 in. for removal of A3 and C1, respectively. As in the case of the previous frames, catenary action is not occurring in these scenarios.

Maximum overall strain rates when column A3 is compromised were also evaluated. The peak strain rate resulting from axial load was on the order of 190 $\mu\varepsilon$/s for the girders and 3,400 $\mu\varepsilon$/s for the columns. Peak strain rates resulting from transverse shear were 1,200 $\mu\varepsilon$/s and 2,400 $\mu\varepsilon$/s for the columns and beams, respectively. Peak strain rates that resulted from bending moments were on the order of 5,500 $\mu\varepsilon$/s for the beam members and 1,700 $\mu\varepsilon$/s for the columns. The strain rates seen in this structure are only slightly more than the intermediate loading rate and orders of magnitude lower than the dynamic loading rate (Barsom and Rolfe, 1999). The loading rates found in the elastic time-history analysis give no indication that fracture toughness of the constituent materials will be insufficient or that the elevated yield strength resulting from increased load rate should be considered. It should be emphasized again that stress raisers caused by connection geometry have been ignored.

One interesting item of note is that because the ground floor columns are rendered ineffective, there is an upward rebound of the floor immediately below the compromised column as its stored strain energy is released. This rebounding effect suggests that moment resisting connections at the ends of beams (be they infill or otherwise) should be designed for full reversal of moments.

The demand-to-capacity ratio (DCR) defined in Equation 2 was used to evaluate members in the 20-story frame. Peak DCR magnitude for columns was 0.82 and peak DCR magnitude for beams was 0.36, indicating elastic behavior. The peak tension forces in the girders immediately adjacent to the compromised column were approximately 40 kips, which is approximately 1.6% of the nominal tension capacity of the beam/girder member. It appears that expressing the tie force as a function of the nominal axial tension capacity of the beam/girder member is relatively consistent across all three frames: 1.9% for the three-story frame; 2.0% for the

10-story frame; and 1.6% for the 20-story frame. It should be emphasized that the three-story frame responded in an inelastic manner to the compromising event whereas the 10- and 20-story frames responded in an elastic manner.

CONCLUDING REMARKS AND RECOMMENDATIONS

Three structural steel frames were chosen for detailed evaluation of system response during compromising events. The event selected was interior column removal at the perimeter of the first floor level. Three-dimensional structural analysis models were developed and the presence of the concrete-steel composite deck floor system was simulated using massless X-bracing members within the plane of the floor plate. Column removal scenarios were modeled using time-history functions. Inelastic and elastic time-history analysis of the 3D systems was used as required by a series of analyses with systematically increasing complexity.

The inelastic analysis results for the three-story frame indicated that the floor system becomes an active participant in load transfer in the response of the system to a compromising event. The diagonal X-bracing members simulating shear racking stiffness of the floor plate were able to preserve the dominate aspects of frame response while preventing undesired removal of axial forces in the main tie members that would result with rigid diaphragm modeling. It also suggests that further study regarding the participation of the concrete-steel composite floor system typically present in the building is warranted and that compartmentalization of damage is likely most effective accomplished within the influence area of the compromised column. Damage compartmentalization may involve one or more floors. As the number of stories decreases, the number of sacrificed floors may need to increase.

It is recommended that all moment-resisting connections be designed for equal moment magnitudes in positive and negative bending. The structural analysis also suggested that full moment reversal at plastic moment capacity magnitudes is likely in the beam and girders in a framework in the event a column is compromised. Therefore, it is recommended that all moment-resisting connections be designed for fully plastic moment capacity in both loading directions.

Strain rates seen in the frames studied were of intermediate magnitude. Therefore, it is expected that elevated levels of yield stress and reduced fracture toughness may not need to be considered in the analysis. However, it should be emphasized that geometric effects that will be present in the connections of the system were not considered. It is recommended that the demand-to-capacity ratios computed in the present study be used to further examine detailed connection response.

If the members in the framework respond in a predominantly elastic manner, it is likely a result of a significant number of floor levels above the compromised area acting as a deep beam. This will be exhibited by beams at several levels above the compromised column being subjected to compression forces and the girders immediately adjacent to the compromised column being subjected to a relatively large tension force. When few floors are present above the compromised column (e.g., the three-story building) an inelastic flexural mechanism is likely to form. The axial tension forces in the girders appear to be independent of the presence of inelastic behavior. The inelastic response of the three-story frame indicated that the girder tension forces were approximately 1.9% of the nominal tension capacity of the girder. The elastic response of the 10-story and 20-story frame suggested a tension tie force of 2.0% and 1.6%, respectively. Therefore, if one tie force magnitude were to be specified as a minimum general structural integrity criterion, it is best to have this as a percentage of the tension capacity of the beams or girders. Two percent of the axial tension capacity (yield) appears reasonable.

The axial, transverse shear, and bending moment demands and application rates for these forces taken from the time-history response traces can be used as the formulation bases for analytical and experimental testing efforts to evaluate detailed connection response. The rotational demands seen in the inelastic analysis of the three-story frame can also be used as initial demand targets for this experimental and analytical work. For example, if one were to consider the DCRs in Table 1, a testing protocol for fully restrained moment connections could be developed as follows: Apply 2% of the tension (yield) capacity of the beam member simultaneously with the fully plastic moment capacity of the connection, and impart rotational demand of 0.022 rad in both positive and negative directions. The present analysis effort can serve as the basis for establishing experimental demand expectations and experimental protocols to be used to study a wide variety of connections in the structural steel framing system and conduct detailed evaluation of the connection response.

Finally, 3D response simulation should be carried out in order to evaluate the inherent structural integrity or robustness present in the structural steel framing system. This 3D response should consider connection characteristics and modeling of the concrete-steel floor slab system. It is very important that studies be undertaken to evaluate other column loss scenarios within this 3D environment to fully assess the tendency for initiation of progressive collapse and the formulation of general structural integrity provisions. Furthermore, situations where there are no floors above the compromised column should be studied as these may be one of the more critical compromising scenarios. This aspect is one focus of the companion paper (Foley et al., 2008).

It should be emphasized that this paper considered a single damage scenario (loss of exterior column). This scenario is but one of many that should be used to assess the inherent structural integrity in the steel framing system.

Part 2 of the two-part article considers additional damage scenarios occurring within the floor framing system. Taken together, these constitute a systematic effort to assess the inherent structural integrity in steel framing systems. It should be noted, however, that additional damage scenarios (e.g., loss of corner column) and the residual capacity of members likely to be present after localized damage occurs should be considered before the book is closed. The current article provides a first-draft roadmap for this activity.

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