Design of Steel Structures for Blast-Related Progressive Collapse Resistance

by Ronald Hamburger, S.E. and Andrew Whittaker, S.E.

Research is needed to develop new connection technologies to make blast-resistant designs more efficient.

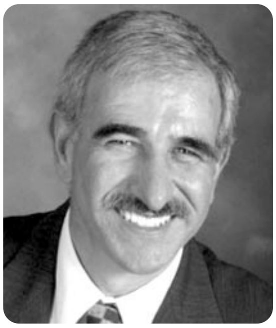
Many government agencies and some private building owners today require that new buildings be designed and existing buildings upgraded to resist the effects of potential blasts. While it is possible to design buildings to resist such attacks without severe damage, some blast-resistance design measures result in unacceptable high costs and architectural limitations. Since the probability that any building actually will be subject to such hazards is low, a performance-based approach to design has evolved. A common goal is to permit severe damage should blasts occur, but avoid massive loss of life. These goals are similar to the performance goals inherent in seismic design, and some federal guidelines for designing blast-resistant structures draw on material in performance-based earthquake-resistant design guidelines. While there are similarities between earthquake-resistant and blast-resistant design, there are also important differences.

Blast-resistant design typically focuses on several strategies including: adequate standoff and access control to limit the approach and entrance of weapons; exterior cladding and glazing systems that avoid generating glazing projectiles in occupied spaces as a result of specified blast-impulsive pressures; and design of structural systems such that loss of one or more vertical load-carrying elements will result in only limited, localized structural collapse. Although blast pressures can be several orders of magnitude larger than typical design wind-loading, the duration of these impulsive loads is so short that they typically are not capable of generating sufficient lateral response in structures to trigger lateral instability and global collapse. Steel structures with complete lateral force-resisting systems capable of resisting wind and seismic loads specified by building codes generally will be able to resist credible blast loads without creation of lateral instability and collapse. However, explosive charges detonated in close proximity to structural elements can cause extreme local damage, including complete loss of load-carrying capacity in individual columns, girders and slabs. Consequently, structural design of steel structures for blast resistance typically is focused on design of vulnerable elements, such as columns, with sufficient toughness to avoid loss of load-carrying capacity when exposed to a small charge, and designing structural systems that are capable of limiting or arresting collapse induced by extreme local damage.

Steel building systems are ideal for this application due to the toughness of structural steel as a material and the relative ease of designing steel structures with adequate redundancy, strength and ductility to redistribute loads and arrest collapse. However, it is essential to create low-cost design strategies for collapse resistance with minimal architectural impact and to demonstrate the effectiveness of technologies for collapse resistance.

Design Strategies

Design strategies for collapse-resistant buildings involve removal of one or more vertical load-carrying elements and demonstrating that not more than specified portions of the building will be subject to collapse as a result. The element removal could occur following loading



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Collapse of World Trade Center 6, induced by falling debris from the North Tower. Note that the dark lines indicate approximate locations of one-bay steel moment frames around building perimeter.

events like blast, vehicle impact, or fire. The design strategy can be traced to lessons learned from the blast-induced collapse of the Alfred P. Murrah Building in Oklahoma City, where extreme damage to columns at the first story led to progressive collapse of most of the structure.

The ASCE investigating team concluded that, had the building been designed with the continuity of structural systems typically present in buildings designed for seismic resistance, the extent of building collapse following blast-induced failure of several first-story columns would have been reduced substantially.

Moment-resisting steel frames are ideal for this continuity and in avoiding progressive collapse. Three examples of their effectiveness can be observed in the performance of the World Trade Center buildings following the terrorist attacks of Sept. 11, 2001. The closely spaced columns and deep girders of the moment-resisting steel frame that formed the exterior wall of the North Tower bridged around the massive local damage caused by impact of the aircraft, and arrested global collapse of the structure for nearly two hours. During the Sept. 11 attack, despite the fact that an entire column was removed from the Deutsche Bank Building over a height of more than 10 stories, its conventional moment-resisting steel frame helped arrest partial collapse from falling debris of the South Tower of the World Trade Center. The WTC-6 building faced the collapse of the North Wall of the North Tower across its top. A series of one-bay moment-resisting steel frames placed around the perimeter of WTC-6 limited collapse to areas not protected by moment-resisting framing.

A building with a continuous moment-resisting steel frame on each line of columns can resist collapse through redistribution of load to adjacent columns. The U.S. General Services Administration developed simplified guidelines for the design of such systems (ARA, 2003), which are available to designers engaged in the design or review of federal facilities. These guidelines specify that elements of the frame be proportioned with sufficient strength to resist twice the dead load and live load anticipated to be present, without exceeding inelastic demand ratios obtained from the federal guidelines for seismic rehabilitation of buildings (ASCE, 2002). The design model utilized in these simple procedures is conceptually incorrect, but probably provides adequate design solutions.

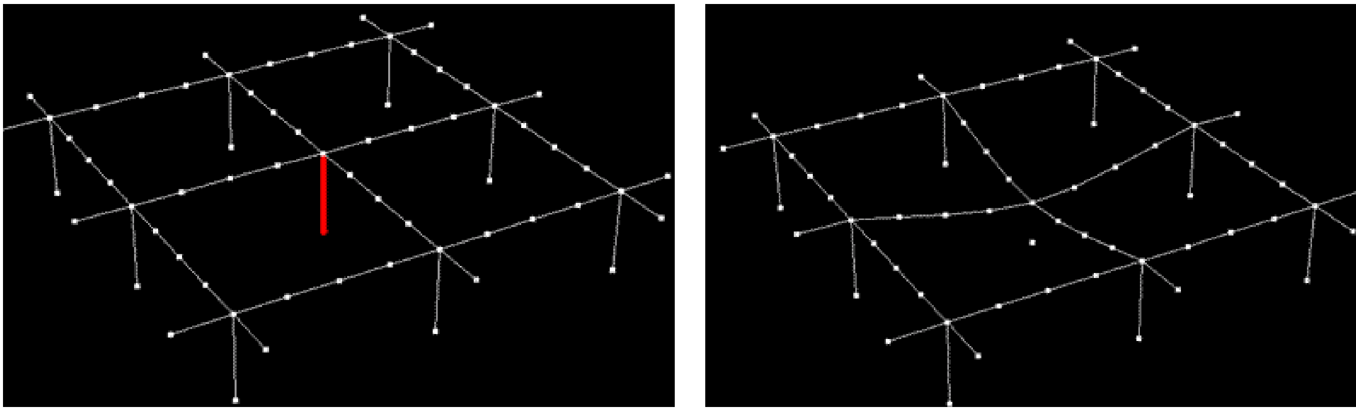
Under this design model, the beams and columns are assumed to distribute twice the vertical forces initially resisted by the removed element through flexural behavior. The elements must be proportioned to resist twice the load initially resisted by the "removed" element, based on theory related to the instantaneous application of load on an elastic element. For example, a structure has a natural period of vibration of 0.5 seconds, a stiffness of 100 kips/inch and moderate damping. A 100-kip load is applied instantaneously to the structure. In response, the structure experiences an instantaneous deflection of 2", then oscillates with slowly decaying amplitude until a steady state deflection of 1" is approached. The maximum force in the structure is 200 kips, or twice the statically applied amount, and the maximum deflection of the structure is 2", or twice the static value, resulting in the impact coefficient of 2 used in the federal progressive-collapse design guidelines.

Under federal progressive-collapse design guidelines, members are permitted to experience "flexural inelasticity" based on permissible values contained in seismic guidelines, recognizing that the amplified loading occurs for a very short duration, and that long-term loading following removal is a static condition. Specifically, compact framing is considered acceptable if the ratio of moment computed from an elastic analysis (M_A) to the expected plastic moment capacity of the section (M_{PE}), is less than 3. Non-compact sections are permitted with a limiting ratio M_A/M_{PE} of 2.

A basic flaw in the federal progressive collapse guidelines is that, while it should be permissible to permit inelastic deformation of framing used to resist collapse, as measured by the M_A/M_{PE} ratio, the structure must, as a minimum, have sufficient plastic strength to support the weight of the structure in a static condition. The federal progressive collapse guidelines do not require this evaluation but should.

A Catenary Alternative

Fortunately, the assumption that load redistribution occurs through flexural behavior alone is conservative and results in the design of members that are larger than required to resist progressive collapse. An alternative load-resisting mechanism relies on catenary behavior of the steel framing and compressive arching of the concrete floor slab. The frame supports loads prior to column removal, and if the central column is removed beneath the floor, the frame redistributes loads to the outer columns through flexure, as the floor locally falls downward. If the girders are not strong enough to resist the strength demands of the instantaneous removal of the central support column in an elastic manner, which is what federal guidelines assume, plastic hinges will form at the two ends of the beams and in the mid-span region, near the removed column. Neglecting loading along the beam span, the two-span beam will have a strength equivalent to $8M_p/L$ (where M_p is the plastic moment capacity of the beam and L is the distance between the outer columns) to resist the load imposed on the beam by the now discontinuous central column, and to slow the downward movement of the floor system. If the strength is insufficient, the beam will deflect enough to mobilize catenary tensile action, which eventually will arrest the collapse if strong enough. This mode of behavior is not considered explicitly in the federal guidelines, but is relied upon. If the beam were compact, and laterally supported, the federal guidelines would permit the beam to arrest the collapse of a central column load with a magnitude as high as $12M_p/L$. In such a case, even though neglected by the federal guide-



Redistribution of gravity loads from removed column in building with a continuous moment-resisting steel frame along column lines.

lines, either catenary tensile behavior will be mobilized or the structure will collapse.

Most designs currently neglect the ability to develop catenary behavior and rely solely on the flexural mechanism. To illustrate the potential efficiency of the catenary mechanism, in a recent study, it was determined that, for a structure with 30′ bay spacing, ASTM A992 W36 horizontal framing safely could support the weight of nearly 20 stories of structure above in the event of column removal, although deflection would be significant. There are several potential implications of this finding. First, it is not necessary to provide moment-resisting framing at each level of a structure to provide progressive-collapse resistance. Second, it is not necessary to have substantial flexural capacity in the horizontal framing, either in the beam section itself or in the connection, to provide collapse resistance. Third, it might not be necessary to provide full moment resistance in the horizontal framing, and conventional steel framing might provide progressive-collapse resistance as long as connections with sufficient tensile capacity to develop catenary behavior are provided.

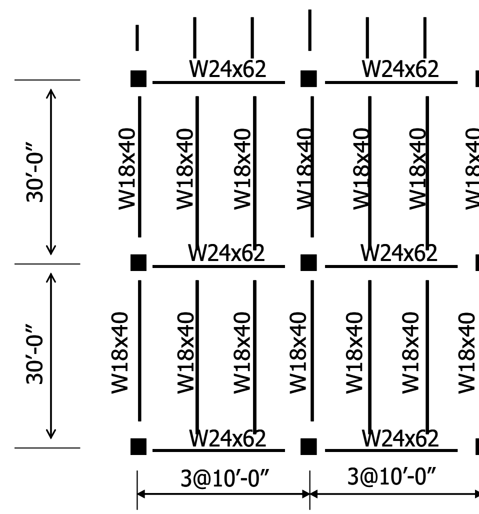
Design Applications

The efficiency of moment-resisting steel frame structures in progressive-collapse resistance was demonstrated in a study of the cost premium associated with providing progressive collapse resistance in a typical structure. A structure with a regular 30′ grid pattern was reviewed. The floor system comprised a 3″, 20-gauge metal deck, supporting a 5½″(total thickness) lightweight concrete slab, with non-composite floor beams. Initially framing was designed without moment-resistance. The resulting framing used W18×40 A992 beams and W24×62 A992 girders. Next, the beams and girders along column lines were assumed to have moment-resistance. An evaluation of the structure for ability to resist instantaneous removal of a single interior column was performed using the federal progressive-collapse guidelines. It was determined that the maximum value of M_A/M_{PE} in the framing was 1.5, or half the permissible value for compact sections. Thus, progressive-collapse resistance can be achieved in steel moment-frame structures without increasing the framing weight.

Nonetheless, there is a significant cost premium with creating moment connections between every beam, girder and column. An additional study was performed to determine if the number of moment-resisting connections in the building could be reduced. First, it was determined that if the moment-resistance was not provided for the W18×40 beams on the column lines but was provided for the W24×62 girders, the maximum value of M_A/M_{PE} was increased only to 1.9, which is well within the limits permitted by the guidelines. Next, researchers determined if it would be possible to create the desired collapse resistance by providing moment resistance on only a few of the floors in a multistory building. It was determined that by using W36×300 sections as the beams and girders at one floor level, it would be possible to provide progressive-collapse protection for as many as 15 supported stories. This results in few moment connections and a total increase in framing weight of about 1.5 pounds per square foot, demonstrating that economical solutions for providing collapse resistance in steel structures is possible.

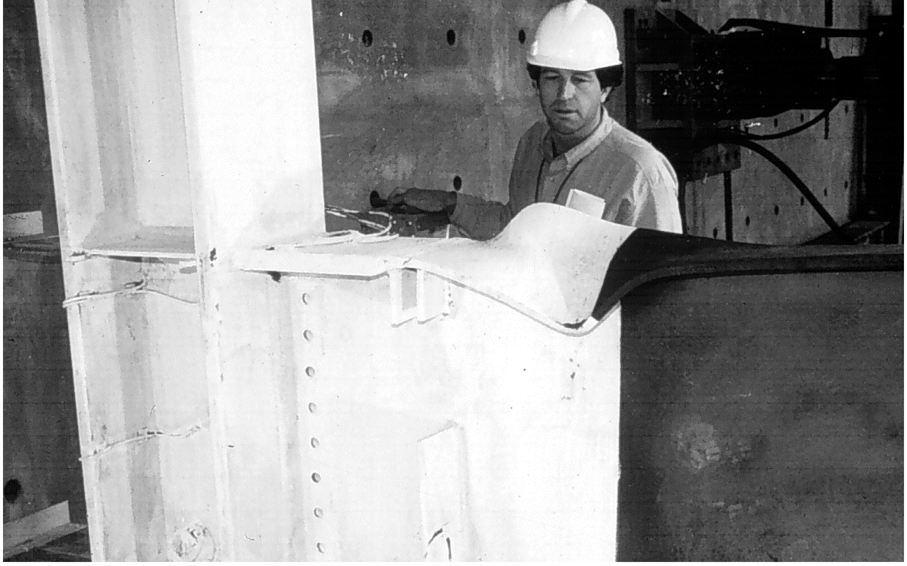
Research Needs

While the use of catenary behavior to provide progressive-collapse resistance holds promise for steel structural design, it is not apparent what types of connections of beams to columns possess suffi-



Typical floor framing evaluated for collapse resistance.

cient robustness to permit the development of plastic rotations at beam ends together with large tensile forces. Mobilization of catenary action in framing could require plastic rotations on the order of 0.07 radians or more. There are substantial differences in the loading demand that occurs on beam-column joints in an earthquake compared to those in a frame-resisting progressive collapse. Earthquake demands are cyclic and induce low-cycle fatigue failure of connections. However, demands applied on members and connections when resisting direct air-blast loadings can produce high strain rates, perhaps of larger magnitude, and will occur simultaneously with large axial tension demands. Under conditions of high strain rate, steel framing becomes stronger but more brittle. There is evidence that standard beam-column connection framing is vulnerable to such loading. In the Deutsche Bank building, the beam that connected to the column using a bolted-flange-plate-type connec-



Extreme plastic deformation of beam-column connection designed for enhanced inelastic behavior

tion was sheared directly off the column due to the impact of debris falling onto the structure from the adjacent collapsing South Tower of the World Trade Center. Failures such as this indicate that standard connection types used in steel framing might not be capable of allowing the structure to develop the large inelastic rotations and tensile strains necessary to resist progressive collapse through large deformation behavior. Regardless, it is known that when properly configured and constructed using materials with appropriate toughness, steel connections can provide outstanding ductility and toughness.

Following the 1994 Northridge earthquake, an extensive program of investigation was undertaken to develop beam-column connections capable of providing reliable behavior under the severe inelastic demands produced by earthquake loading. A number of connection configurations capable of acceptable behavior were developed (SAC 2000a). In parallel with these connection configurations, a series of materials, fabrication and construction-quality specifications also were produced (SAC 2000b).

While these technologies have been demonstrated capable of providing acceptable seismic performance, it is unclear whether they are appropriate for protection against progressive collapse. Some of the connection configurations presented rely on relief of high stress and strain conditions in the beam-column connection through intentional reduction in cross section that could lead to other failures under high-impact load conditions. However, it is also possible that less robust connections than those necessary for seismic resistance could be adequate to arrest collapse in some structures.

The moment-resisting connections in the WTC 6 building, for example, which were not particularly robust by seismic standards, were able to successfully arrest collapse of that structure.

Designers urgently need a program of research and development similar to that conducted after the 1994 earthquake to determine the types of connection technologies that can be effective in resisting progressive collapse so that less conservative but more reliable approaches to blast-resistant design can be adopted by the community.

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This paper has been edited for space considerations. To learn more about blast-resistant design, read the complete text online at ***www.modernsteel.com*** *or in the 2004 NASCC Proceedings.*