Alternative Methods of Evaluating and Achieving Progressive Collapse Resistance

RONALD HAMBURGER, S.E.[1]

ABSTRACT

Structural steel framing is an excellent system for providing building structures the ability to arrest collapse in the event of extreme damage to one or more vertical load carrying elements. The most commonly employed strategy to provide progressive collapse resistance is to use moment-resisting framing at each floor level to re-distribute loads away from failed elements to alternative load paths. Design criteria commonly employed for this purpose typically rely on the flexural action of the framing to redistribute loads and account for limited member ductility and overstrength using elastic analyses to approximate true inelastic behavior. More efficient design solutions can be obtained by relying on the development of catenary behavior in the framing elements and some designs have relied totally on catenary mechanisms. However, in order to reliably provide this behavior, steel framing connections must be capable of resisting large tensile demands simultaneously applied with large inelastic flexural deformations and the structure as a whole must be capable of distributing these large tensile demands through a complete load path. Research is needed to identify framing connection technologies capable of reliable service under these conditions.

INTRODUCTION

Many government agencies and some private building owners today require that new buildings be designed and existing buildings evaluated and upgraded to provide an ability to resist the effects of potential blasts and other incidents that could cause extreme local damage without sustaining large scale collapse. While it may be possible to design buildings to resist such attacks without sustaining extreme damage, the loading effects associated with these hazards are so intense that design measures necessary to provide such performance would result in both unacceptably high costs as well as impose unacceptable limitations on the architectural design of such buildings. Fortunately, the probability that any single building will actually be subjected to such hazards is quite low. As a result, a performance-based approach to design has evolved. The most common performance goals are to permit severe and even extreme damage should blasts or other similar incidents affect a structure, but avoid massive loss of life. These goals are similar, though not identical to the performance goals inherent in design to resist the effects of severe earthquakes, and indeed, some federal guidelines for designing collapse-resistant structures draw heavily on material contained in performance-based earthquake-resistant design guidelines. While there are many similarities between earthquake-resistant design and collapse-resistant design, there are also important differences.

The basic principles of earthquake-resistant design include: providing structures with sufficient lateral strength and stiffness to withstand the effects of horizontal ground shaking without excessive lateral displacement and development of instability; providing continuous load paths throughout the structure, such that it responds to shaking as an integral unit; tying all portions of the structure together, so that components are not shaken loose, and detailing primary load carrying elements so that they can sustain large inelastic deformations without loss of load carrying capability. Earthquake-induced displacements, though large when compared with typical displacements experienced under dead and live loads, are actually quite limited. Lateral displacements due to earthquake are generally on the order of a few inches per story or less. Rotations induced on beams and columns are generally on the order of a few hundredths of radians. Structural response to ground shaking is cyclic in nature and buildings responding to earthquake motion may experience a number of cycles of large displacement motion during a large magnitude earthquake. However, once the earthquake ends, the structure comes to rest, and stresses imposed by lateral shaking are typically relieved.

Design for collapse resistance is typically accomplished by providing alternative load paths to resist gravity loads, in the event that one or more primary gravity load bearing elements are compromised or destroyed. Like design for earthquake resistance, continuous and redundant load paths are essential to accomplish the load re-distribution

[1] Principal, Simpson Gumpertz & Heger, Inc., San Francisco, CA

required to re-distribute gravity loads under such conditions. Also, as with design for earthquakes, it is typically uneconomical to design structures to remain elastic during these loading conditions, and so, it is important to detail structures such that they can experience inelastic deformation without loss of load carrying capability. Although the inelastic loading experienced during the arrest of a collapse is not generally cyclic in nature, it is caused by gravity loading and therefore, is a sustained, rather than short duration loading. Finally, the vertical deformations experience by a structure while arresting collapse can be much larger than those sustained by structures responding to earthquakes.

Steel building systems are ideally suited to design for collapse resistance. The toughness of structural steel as a material, and the relative ease of designing steel structures such that they have adequate redundancy, strength and ductility to redistribute loads and arrest collapse, facilitate the design of collapse-resistant steel structures. However, effective design strategies that will provide collapse resistance at low cost and with minimal architectural impact are urgently needed as is research necessary to demonstrate the effectiveness of technologies employed to provide the desired collapse resistance. This paper explores these issues.

DESIGN STRATEGIES

Typical approaches for collapse-resistant building design involves demonstration that not more than specified portions of a building will be subject to collapse if the gravity load carrying capability of one or more vertical load carrying elements is suddenly lost. The initial loss of load carrying capability could be the result of an explosion, vehicle impact, fire or other cause. The actual cause of the initial damage to the gravity load-bearing system is typically not specified in the design procedure however, the damage is assumed to be sudden and permanent. The engineer must determine that once the hypothetical damage has occurred, the structure is capable of redistributing the gravity loads, through alternative load paths and that collapse does not progress beyond certain specified limits. The basis for this design approach can be traced to lessons learned from observation of the blast-induced collapse of the Alfred P. Murrah Building in Oklahoma City on April 19, 1995. As illustrated in Figure 1 (Partin 1995) extreme damage to columns at the first story of the 9-story building, led to progressive collapse of much of the structure (Figure 2).

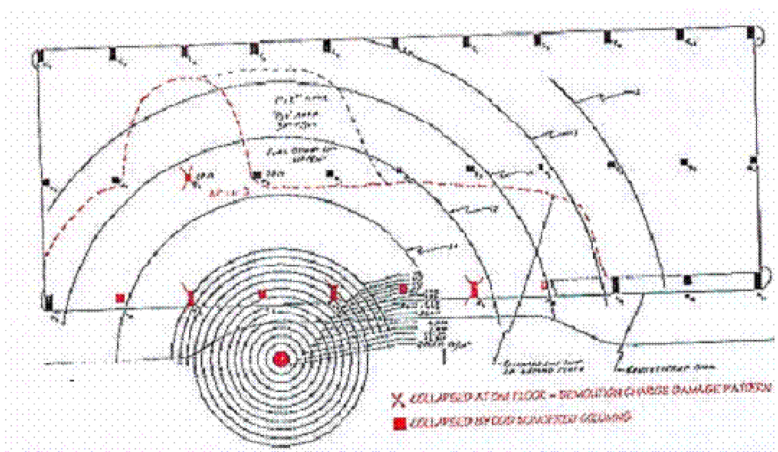

Figure 1. Diagram showing elements damaged by initial blast adjacent to Murrah Federal Building

In their report on the performance of the building, the ASCE investigating team (Sozen 1995) speculated that had the building been designed with the continuity and ductility of structural systems typically present in buildings designed for seismic resistance, the extent of building collapse following blast-induced failure of several 1st story columns would have been substantially reduced.

Figure 2. Remains of the Murrah Building after blast-induced progressive collapse

Moment-resisting steel frames are ideally suited to provision of this continuity and ductility necessary to avoiding progressive collapse. Three examples of the effectiveness of moment-resisting steel frames in arresting collapse and preventing progressive collapse as a result of extreme localized damage can be observed in the performance of buildings at New York's World Trade Center following the terrorist attacks of September 11, 2001. Figure 3 is a view of the north face of the North Tower of the World Trade Center, clearly indicating that the closely spaced columns and deep girders of the moment-resisting steel frame that formed the exterior wall of the structure were capable of bridging around the massive local damage caused by impact of the aircraft and thereby arrest global collapse of the structure for nearly 2 hours. Figure 4 illustrates that the more conventional moment-resisting steel frame of the Deutsche Bank Building allowed that structure to arrest partial collapse induced by falling debris from the south tower of the World Trade Center, despite the fact that an entire column was removed from the structure over a height of 10+ stories. Figure 5 is a plan view of the WTC-6 building at New York's World Trade Center following collapse of the North Wall of the North Tower across the top of the building. A series of one-bay moment-resisting steel frames placed around the perimeter of the building arrested collapse and limited collapse to areas not protected by moment-resisting framing.

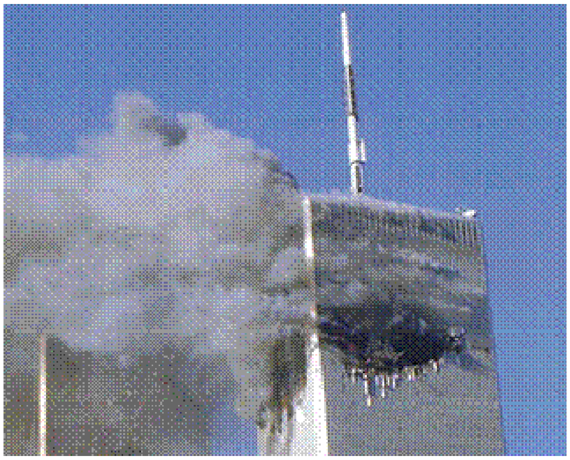

Figure 3. North Tower of World Trade Center, Illustrating the ability of the perimeter frame to bridge around the massive aircraft impact damage and arrest progressive collapse.

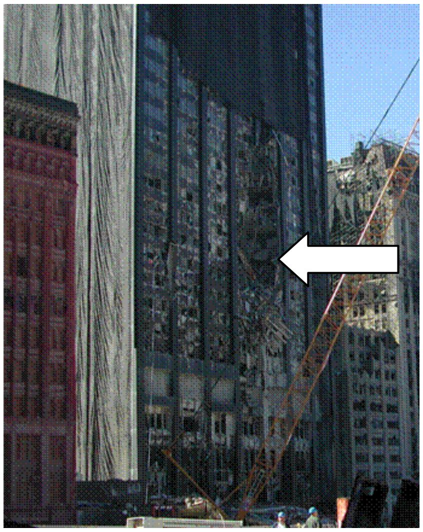

Figure 4. Deutsche Bank Building remains standing despite column loss over multiple stories (see arrow)

Figure 5. 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 the building perimeter.

The use of moment-resisting steel framing to provide collapse resistance is an obvious choice. Figure 6 illustrates how a building with a continuous moment-resisting steel frame on each line of columns can resist collapse through redistribution of load to adjacent columns. Simplified guidelines for the design of such systems have been developed for the U.S. General Services Administration (ARA, 2003) and are available to designers engaged in the design or review of federal facilities. These guidelines specify several procedures to determine the adequacy of a frame to redistribute gravity loads in the event of failure of a primary load-bearing element. Methods permitted by the guidelines include a linear static procedure, a nonlinear static procedure and a nonlinear dynamic procedure. Under the linear static procedure, the elements of the frame are proportioned with sufficient strength to resist twice the dead and live load anticipated to be present, without exceeding permissible inelastic demand ratios obtained from the federal guidelines for seismic rehabilitation of buildings (ASCE, 2002). Under this design approach, the beams and columns are assumed to distribute vertical forces initially resisted by the removed element, through flexural behavior. The elements are proportioned to resist twice the load initially resisted by the "removed" element as the peak demand on an elastic member when load is instantaneously placed on it will be twice the static value. Members are allowed 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. As with the federal seismic rehabilitation guidelines, there is an inherent assumption that demand to capacity ratios, calculated with a linear model provide reasonable indications of inelastic ductility demands. The GSA Progressive Collapse guidelines also permit the use of a nonlinear static, or “pushdown” procedure, in which the a nonlinear model of the structure is allowed to progressively yield until it is demonstrated that the structure is capable of sustaining twice the applied dead and live loads. The nonlinear dynamic procedure uses dynamic nonlinear finite element analysis to directly predict the stress distribution resulting from element removal as a function of time.

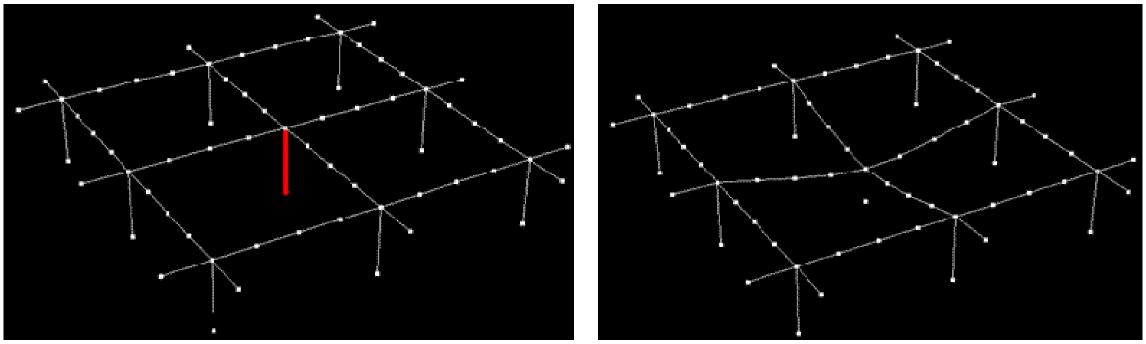

Figure 6. Redistribution of gravity loads from removed column in building with a continuous moment-resisting steel frame along column lines.

The assumptions underlying both the linear static and nonlinear static procedures contained in the GSA Progressive Collapse Guidelines are incorrect and as a result, these procedures can lead to inappropriate design solutions. A primary problem with both procedures is the assumption that a load increase factor of 2 adequately accounts for the impact effects associated with instantaneous application of load to a structure. It is well known when a load is instantaneously applied to a structure that remains elastic, the peak stress and displacement of the structure will be twice the values associated with static loading. However, this rule does not apply for structures that experience inelastic straining in response to the instantaneously applied load. Nonlinear dynamic analysis of a large number of single degree of freedom systems, having different natural periods of vibration and different levels of strength clearly show that when the yield strength of a structure is less than 150% of the strength required to resist the statically applied load, the peak displacements experienced under instantaneous load application will be larger than 2. This affect appears to be largely independent of the natural frequency of the system for the range of frequencies typically found in building systems.

Figure 7 illustrates an approximate relationship derived from such dynamic analyses. The vertical axis of the figure is the ratio of the peak displacement experienced by the structure, upon instantaneous load application. The horizontal axis is the strength ratio, which is the yield strength of the structure divided by the instantaneously applied load. Values of the strength ratio in excess of 2 represent elastic response, and as can be seen in the figure, result in displacement ratios of 2. The displacement ratio of 2 remains valid at strength ratios in excess of about 1.6. At strength ratios below 1.5, the displacement ratio increases exponentially and as the strength ratio approaches unity, becomes infinite. Thus, it can be seen that the load factor of 2, used by the static procedure in the Progressive Collapse Guidelines can severely underestimate the deformation demand on the structure if it is not capable of redistributing the load in a nearly elastic manner.

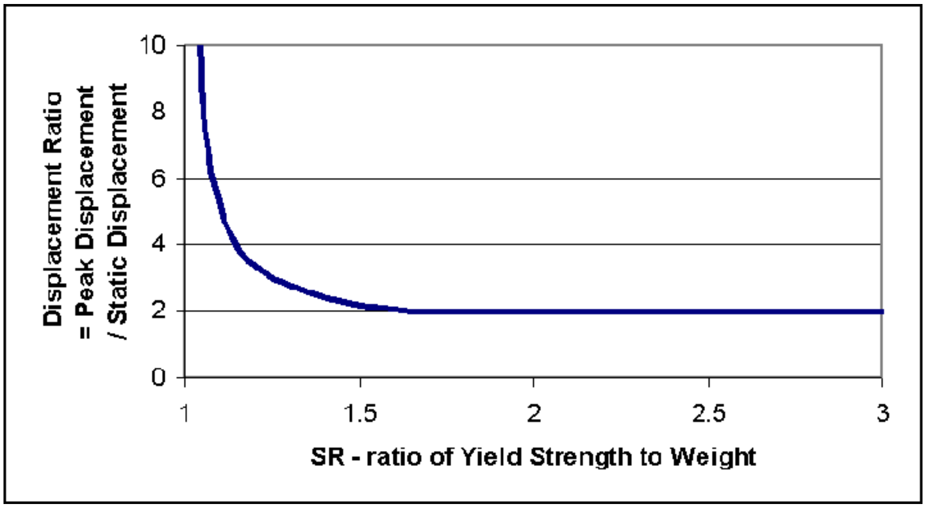

Figure 7 – Impact factor for instantaneously applied loads to inelastic systems

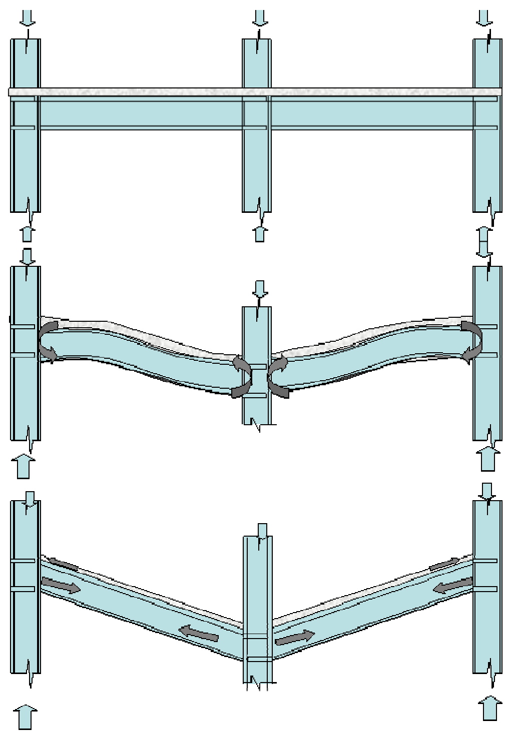

Figure 7. Load-resisting mechanisms upon removal of a supporting column

Most designs presently neglect, at least explicitly, the ability to develop catenary behavior and implicitly rely solely on the flexural mechanism. As an illustration of the potential efficiency of the catenary mechanism, in a recent study, it was determined that in a structure with 30 foot bay spacing, ASTM A992, W36 horizontal framing could safely support the weight of nearly 20 stories of structure above in the event of column removal (Hamburger, 2003), 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, in order 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, in order to provide this collapse resistance. Third, it may not be necessary to provide full moment resistance in the horizontal framing and conventional steel framing may be able to provide progressive collapse resistance as long as connections with sufficient tensile capacity to develop catenary behavior are provided.

RESEARCH NEEDS

While the use of catenary behavior to provide progressive collapse resistance holds great promise for steel structure design, it is not immediately apparent what types of connections of beams to columns will possess sufficient robustness to permit the necessary development of plastic rotations at beam ends together with large tensile forces. Figures 8 and 9 are pictures of bolted web–welded flange moment resisting connections that fractured in the 1994 Northridge earthquake. These fractures occurred in beam column joints under estimated drift demands of approximately 0.01 radian corresponding approximately with the yield rotation capacity of the assembly.

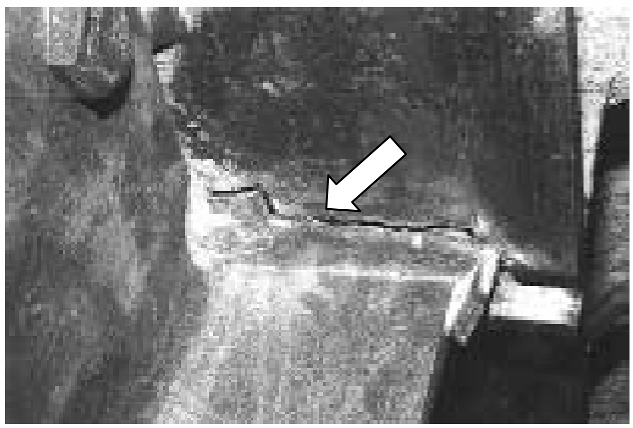

Figure 8. Fractured welded beam flange to column flange connection in building discovered following the 1994 Northridge earthquake

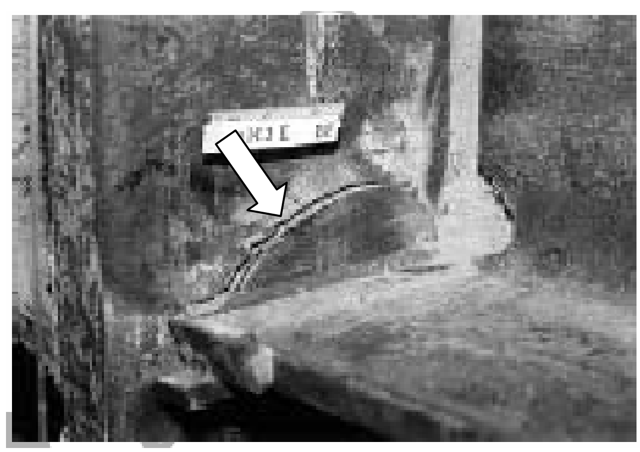

Figure 9. Fractured welded beam flange to column flange joint discovered following the 1994 Northridge earthquake.

Mobilization of catenary action in framing may require plastic rotations on the order of 0.07 radians or more. It is true that there are substantial differences in the loading demand that occurs on beam-column joints in an earthquake as compared to those that occur 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 the initiation of a collapse may produce somewhat higher strain rates, may be of larger magnitude and will occur simultaneously with large axial tension demands. Under conditions of high strain rate, steel framing becomes both stronger but more brittle. There is evidence that standard beam-column connection framing is quite vulnerable to such loading. Figure 10 is a photograph of a failed beam-column connection in the Deutsche Bank building. The beam which connected to the column using a bolted flange plate type connection 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. Also visible at the bottom of this picture is failure of the bolted column splice. Figure 11 is a picture of a failed bolted shear connection in the World Trade Center 5 building that resulted from development of large tensile forces in the beam due to fire effects. Clearly, these failures indicate that standard connection types commonly used in steel framing today may 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. Despite these poor behaviors, it is also known that when properly configured and constructed, using materials with appropriate toughness, steel connections can provide outstanding ductility and toughness. Figure 12 illustrates the deformation capacity of beam-column connections designed with appropriate configurations and materials.

Figure 10. Failed beam-column connection in Deutsche Bank Building

Figure 11. Failed bolted high strength shear connection in World Trade Center 5

Following the damage experienced in steel buildings in 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 were also produced (SAC 2000b). While these technologies have been demonstrated capable of providing acceptable seismic performance, it is unclear whether these technologies are appropriate to providing protection against progressive collapse. Indeed, some of the connection configurations presented in the SAC documents 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 demonstrated as necessary for seismic resistance could be adequate to arrest collapse in some structures. The moment-resisting connections in the World Trade Center 6 building, for example, which were not particularly robust by seismic standards, were able to successfully arrest collapse of that structure.

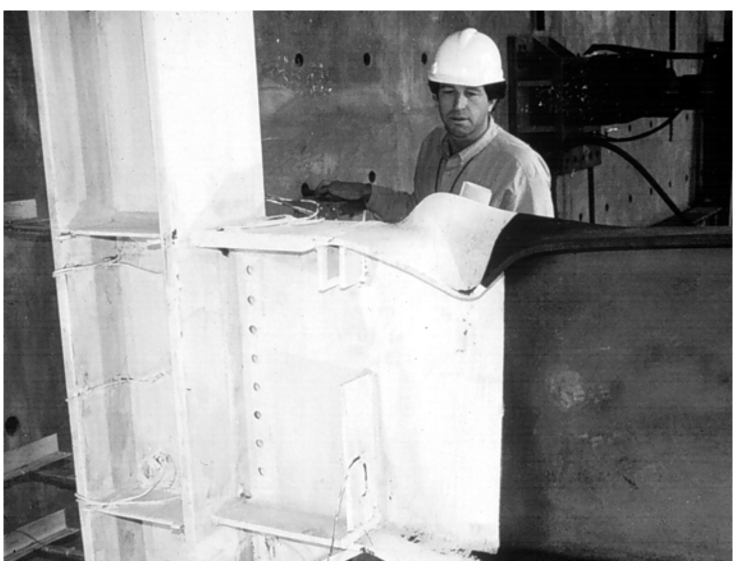

Figure 12. Extreme plastic deformation of beam-column connection designed for enhanced inelastic behavior

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.

REFERENCES

Federal Emergency Management Agency (FEMA). *Recommended Design Criteria for New Steel Moment Frame Construction,* Report No. FEMA 350, prepared by the SAC Joint Venture for FEMA, Washington, D.C., 2000c.

Federal Emergency Management Agency (FEMA). *Recommended Quality Assurance Criteria for Steel Moment Frame Construction,* Report No. FEMA 353, prepared by the SAC Joint Venture for FEMA, Washington, D.C., 2000c.

Federal Emergency Management Agency (FEMA). *Prestandard and Commentary for Seismic Rehabilitation of Buildings,* Report No. FEMA 356, FEMA, Washington, D.C., 2000a.

General Services Administration (GSA). *GSA Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Expansion Projects*, prepared by Applied Research Associates for GSA, Washington, D.C., 2003.

Hamburger, R, Mayes, R, Parker, J. "Impact of High Strength Steel on Building Applications, 3 Applications," August, 2003 Advanced Technology Institute, Proceedings of Workshop on High Strength Low Alloy Steels in Defense Applications, Chicago, IL

Partin, B.K., *Bomb Damage Analysis of the Alfred P. Murrah Federal Building, Oklahoma City, Oklahoma, 1995*, Congressional Register

Sozen, M., Thornton, C., Mlakar, P., Corley, G., "The Oklahoma City Bombing: Structure and Mechanics of the Murrah Building," *Journal of the Performance of Constructed Facilities*, ASCE, Reston, VA, Vol. August, 1998, pp. 120-136