Blast and Progressive Collapse

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INTRODUCTION

The terrorist attacks of 2001 riveted our attention on supposed deficiencies in our structural designs, regardless of the fact that those structures, and the structures surrounding them, actually performed well given the extreme loads to which they were subjected. While these attacks served as a call to action to reevaluate our designs for these severe loads, the fact is that practitioners in the fields of blast loads prediction and dynamic inelastic structural response prediction have been moving steadily forward on research, guideline development and design practices for extraordinary loads such as blast and impact for the past four decades. Granted, these loads and the requisite analysis and designs to resist these loads have not been "textbook" practices in the past. The complexity of the loads and response mechanisms of individual components and assemblages of components has required that the development of the design practice in this field has been one involving a mix of empirical, analytical and, recently, sophisticated numerical methods.

This "Facts for Steel Buildings: Blast and Progressive Collapse" document serves to provide the latest information and guidance available for commercial and industrial buildings subjected to these extraordinary loads and responses. It is not intended to supplant existing guidance for hardened military construction for warfighters. The document presents background and definitions for explosive loads and progressive collapse, general principles of blast loads and response prediction, recommendations for structures designed to resist blast and mitigate progressive collapse, recent guidelines and Federal and DoD requirements, some observations from historical events, and some information on ongoing research.

This document is intended to be a "primer" for engineers, architects, developers and owners. A follow-on and companion "Design Guide for Blast and Progressive Collapse", to be published by AISC, will provide detailed analysis and design recommendations.

This document is presented in 8 sections as follows:

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SECTION 1
GENERAL SCIENCE OF BLAST EFFECTS

1.1. How are explosive and impact loads similar to and different from loads typically used in building design?

Traditional structural system design live loads have been developed through the careful investigation of repetitive events; i.e., loads occurring at regular intervals in time with moderate to severe intensity that are predictable using statistical approaches. Blast events or intentional impact events with extreme severity are loads with very low frequency of occurrence, but with extraordinary consequences.

The closer relationship between blast loads and typical building loads that designers use for building design is one that has come about only in recent years. Designers may now be required to consider loads previously thought rare and improbable, loads caused by intentional attack of our buildings by criminals and terrorists intent on using the failure of our designs to achieve their ends. As stated in ASCE 7-02 Commentary Section C2.5, "Load Combinations for Extraordinary Events":

> "…extraordinary events…should be identified, and measures should be taken to ensure that the performance of key loading-bearing structural systems and components is sufficient to withstand such events."[i]

Figure 1.1 illustrates one such extraordinary event; the horrendous damage caused by a vehicle bomb detonated outside the Oklahoma City Federal Building in 1995.

Explosive loads and impact loads are transients, or loads that are applied dynamically as one-half cycle of a high amplitude, short duration airblast or contact and energy transfer related pulse.

Crowder. 2004

Figure 1.1
Murrah Building, Oklahoma City

This transient load is applied only for a specific and typically short period of time—in the case of blast loads, typically less than one-tenth of a second. This means that an additional set of dynamic structural properties not typically considered by the designer, such as rate-dependant material properties and inertial effects must be considered in design.

Often, design to resist blast, impact and other extraordinary loads must be thought of in the context of life safety, not in terms of serviceability or life-cycle performance. Performance criteria for other critical facilities (nuclear reactors, explosive and impact test facilities, etc.) may require serviceability and reuse, but most commercial office and industrial facilities will not have to perform to these levels. Structures designed to resist the effects of explosions and impact are permitted to contribute all of their resistance, both material linear and non-linear (elastic and inelastic), to absorb damage locally, so as to not compromise the integrity of the entire structure. It is likely that local failure can and may be designed to occur, due to the uncertainty associated with the loads.

1.2. How does a building structure respond to blast loads?

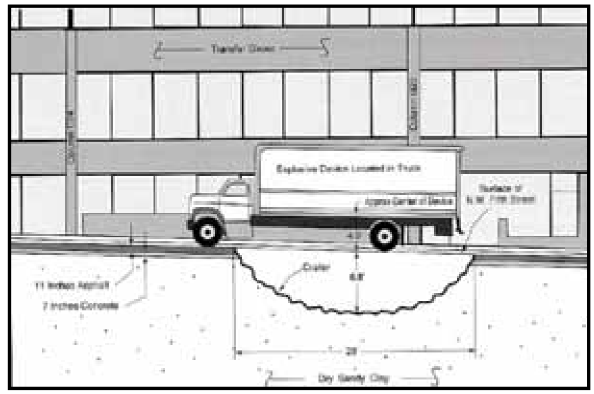
Explosive blast is unlike other types of severe loads caused by extreme events such as earthquake or high wind. These types of loads generate damage that is limited to a very few structural response mechanisms, and they are applied "globally" such that the entire structural system works to resist the load. Explosive blast activates many structural response mechanisms because of its extreme spatial and time variations in magnitude and time of application (duration).

One way to illustrate and describe these multiple, varying, concurrent and sequential mechanisms is to describe the progress of a real event. Figure 1.1 shows the final damage to the Oklahoma City Federal Building. Figure 1.2 (below) shows the building prior to the attack. (The location of the car near the north façade illustrates the short distance between the curb and the transfer girder column line). Figures 1.3 through 1.8 provide schematic views of the damage mechanisms and sequence of the blast loading during the Oklahoma City attack.

FEMA 277, 1996

Figure 1.2
Oklahoma City Federal Building Prior to Attack

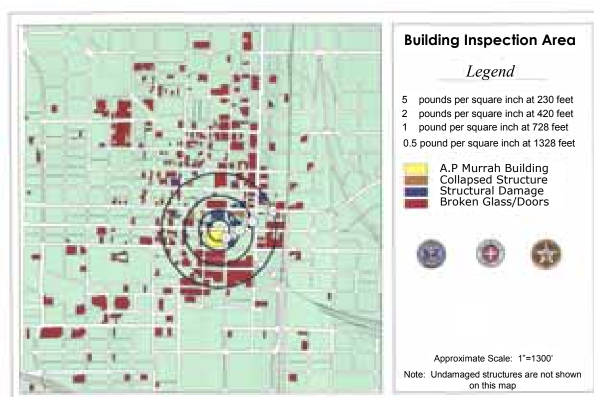
- Crater—Figure 1.3 illustrates the initial and immediate effect generated by the detonation of the approximately 4,000-lbs of improvised explosives used in that attack, the 7-ft deep by 30-ft diameter crater formed in the pavement and subsurface material.



FEMA 277, 1996

Figure 1.3
Explosives-Laden Truck and Crater Formed in Oklahoma City Attack

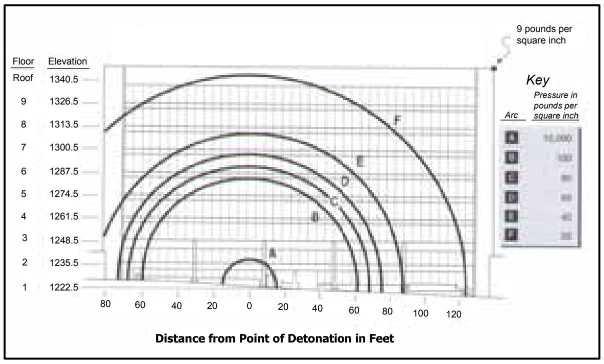
- Extent of Damage—Figure 1.4 illustrates that damage from an explosive charge of this size emanates without respect to direction; collapsed buildings were observed as far as three blocks away and broken glass was observed as far as 4,800 ft away from the blast.



FEMA 277, 1996

Figure 1.4
Blast Overpressure and Extent of Damage in Downtown Oklahoma City

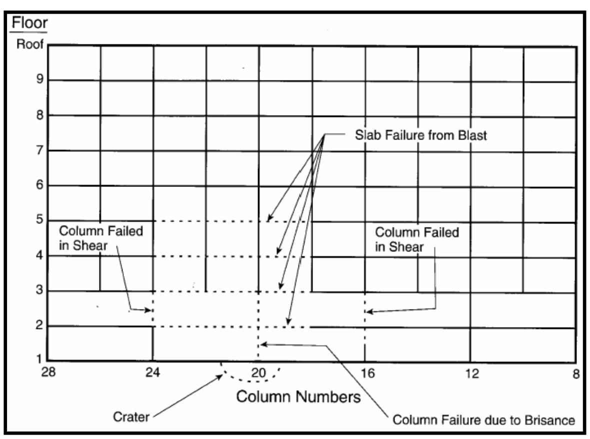
- Direct Blast Loads on Murrah Building—Figure 1.5 shows the distribution of peak blast loads over the building exterior. The dramatic variation of applied pressures can be seen as ranging from approximately 10,000 psi near the columns supporting the transfer girder, to 20 psi at points 120 ft away (parts of levels 7-9 and the roof).



FEMA 277, 1996

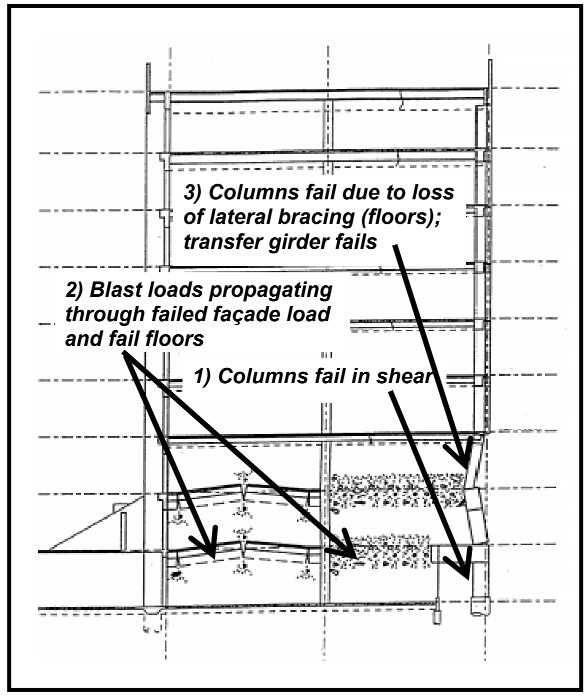
Figure 1.5
Murrah Building Blast Overpressure Distribution

- Blast Load Propagation and Progression of Damage—Figures 1.6 through 1.8 show the propagation of the blast and the effects of that propagation. Figure 1.6 illustrates the failed components on the North elevation. Figure 1.7 illustrates with numbered sequence the damage to the building columns, transfer girder and floors (elimination of lateral support for the remaining columns), while Figure 1.8 shows the position of the damaged elements just before the final collapse.



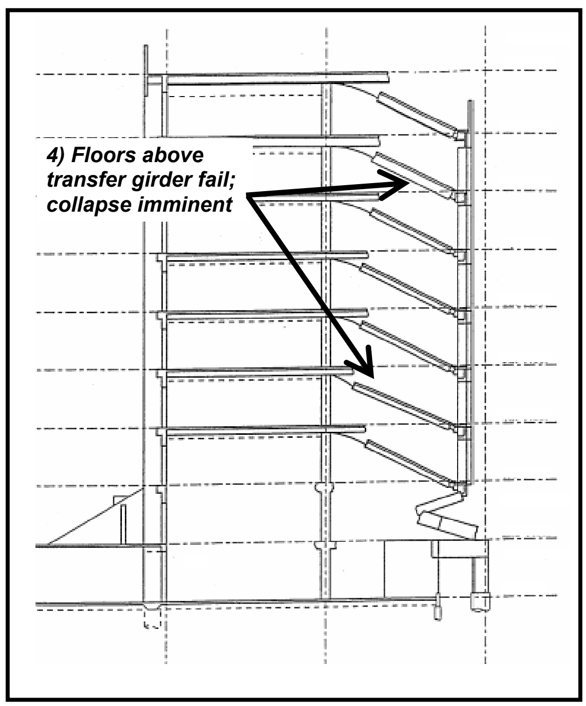
FEMA 277, 1996

Figure 1.6
Failed Components; Murrah Building North Elevation



Woodson, et al, 1995

Figure 1.7
Progression of Loads and Damage in Murrah Building Collapse



Woodson, 1995

Figure 1.8
Pre-Collapse Position of Building Components in Murrah Structure

The findings and conclusions reported in FEMA 277 ("The Oklahoma City Bombing: Improving Building Performance Through Multi-Hazard Mitigation") suggest, as illustrated in the figures, that the vulnerability of the first floor columns and the dependence of the ordinary moment frame on the performance of these columns and the supported transfer girder, led to the collapse of the building.

1.3. Are we "hardening" our building designs or are we protecting people?

Both. The overriding purpose in providing blast resistance is the protection of people. The designer should be familiar with casualty mechanisms that will be made manifest in a blast environment and should prepare a building design that will minimize the effects of these mechanisms. The designer must also be aware of the importance of the egress of survivors and the evacuation of the wounded after an event. It is virtually impossible to prevent casualties in a blast environment. However, a great deal can be done to minimize casualties.

The designer should be aware that casualties in a blast environment are produced by:

1) The interaction of people with energized (penetrating and non-penetrating) debris from the breakup of window glass, exterior and interior walls, etc.

2) The interaction of the blast wave with people. In this interaction the people are tumbled and impact with hard surfaces, attached and unattached objects, and other people. An impact velocity of about 23 ft/sec of the human head with a hard surface results in fatality.

3) The interaction of people with the ground plane if they are ejected from the lower and particularly the upper stories.

1.4. What types of devices can generate blast loads?

Explosive blast loads and shock waves in air that subsequently load structures and objects in their path can be generated by a wide range of devices acting accidentally or used deliberately. Figure 1.9 shows an improvised mortar designed to launch explosive warheads at U.S. assets in Baghdad, Iraq.

Conrath. 2004

Figure 1.9
Improvised Mortar, Baghdad, Iraq

Explosion sources include:

- bursts of pressure vessels with inert, flammable and detonable gases,
- deflagration and detonation of dusts and particulates,
- accidentally or intentionally released flammable or detonable gases in semi-confined and unconfined spaces,
- improvised explosive devices consisting of agricultural fertilizer and diesel fuel, black powder devices and modified military weapons, and
- a wide variety of commercial (blasting) and military explosives

In general, explosions can be thought of as "bomb blasts" resulting from exothermic chemical reactions where energy is released over a sufficiently small time in a sufficiently small volume such that a pressure wave of finite amplitude and length is produced that travels away from the source. So-called ideal explosions are simply those based on a convention assuming energy release at a point. The resulting blast waves can travel in air, which is generally of most interest to building designers, but may also travel in soil (called groundshock) or water.

1.5. What is an explosion, detonation and deflagration?

An explosion is a rapid release of stored potential energy characterized by a bright flash and an audible blast. Part of the energy is released as thermal radiation (flash); and part is coupled into the air as airblast and into the soil (ground) as ground shock, both as radially expanding shock waves. To be an explosive, the material must:

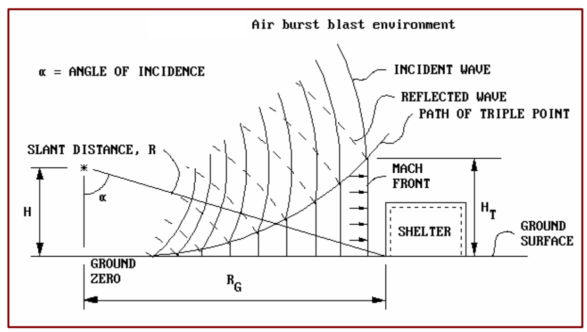
- -contain a substance or mixture of substances that remains unchanged under ordinary conditions, but undergoes a fast chemical change upon stimulation.
- produce a reaction that yields gases whose volume—under normal pressure, but at the high temperature resulting from an explosion—is much greater than that of the original substance.
- produce a change that is exothermic in order to heat the products of the reaction and thus increase their pressure.

Blast loads are most often thought of as emanating from exothermic reactions resulting in “detonations”, however, shock waves in air can result from pressure vessel ruptures and high flame front velocity combustions (typical of unconfined vapor cloud explosions) as well. When the source material can sustain a supersonic wave (flame front) of sufficient velocity to create a local high pressure within the source material, the reaction is considered a detonation, where all stored potential energy is released in the chemical reaction. Flame front velocities in the combustion below this detonation velocity will result in a reaction (an explosion) short of a detonation, called a deflagration, where only a portion of the stored potential energy is released in the chemical reaction.

Often, the class of explosion, be it deflagration or detonation, is determined by the initiation energy available. This initiation energy is usually delivered as a strong shock provided by impact, detonation of a primary explosive, or friction.

1.6. What are the effects the structure must resist?

Shock waves in air decay exponentially with distance. When encountering an obstruction in their path, blast waves will “reflect” to amplitudes many times their “free air” value. This reflection is a function of the strength of the air shock and the angle of incidence of the shock wave front and the structure. When subjected to the shock or blast wave, the structure can respond in several ways, depending on the strength of the blast (a function of the explosive yield or size and the proximity of the structure to the explosion source) and the duration of the wave. Figure 1.10 illustrates the interaction of a blast wave with a structure.



Conrath. 2003

Figure 1.10
Schematic of an Airblast and Structure interaction Environment

The duration of a typical blast pulse is normally a small fraction of the natural period of a building structure and therefore, such loads typically have little significant impact on the overall lateral force resisting systems of buildings. However, the frequencies of individual elements often are in this range and severe local damage to individual elements is likely. When this occurs, of course, it can lead to further instability.

If in a very strong blast wave region, the structure may respond locally with shattering, shearing and tearing of the structural material. This has historically been called the "brisance effect" or very high pressure effect of the explosive. In a region of lower, more uniform pressure, the structure may respond more traditionally; i.e., deformations will be based on flexural response or support shear. Ductility in the structure is important in either case. The optimal structural response to blast is one where elements deform and use up or absorb energy prior to fracture or failure. Figure 1.11 illustrates the surprising ductility observed in steel members in actual blast events.

Crowder. 2004

Figure 1.11
Ductile Column Response to Blast

The goal of blast and impact resistant design is to first prevent the compromise or collapse of the structural system and to maintain structural integrity through ductile and redundant behavior. A secondary but critical goal is to reduce or eliminate debris—which, short of collapse, is the chief cause of casualties and fatalities in explosions. Structural integrity must be maintained even after the local loss of key structural members, thus preventing a disproportionate or progressive collapse of the structure.

If the blast loading is large enough to compromise the vertical load carrying capacity of a column or other vertical load carrying element, post blast development of large tension loads in the beams due to large displacements generated by a double-span condition is likely to occur. In such cases the beams start acting less like beams (i.e., resisting vertical loads through bending) and more like cables, resisting vertical loads through catenary action, resulting in post-yield interaction of moment and axial tension in both the beams and their beam-to-column-to-beam connections.

Floor failure due to infiltration of blast pressures through failed facades (cladding and glazing) could also create double vertical span conditions (doubled unsupported lengths) for columns.

Explosions may also generate primary debris (bomb or vehicle fragments) or secondary debris consisting of blast-borne debris or debris from failed structures near the source. It may be necessary to consider this debris in the structural design to reduce hazards to the occupants and prevent the loss of key structural components. Figure 1.12 shows the results of a U.S. Air Force barracks bombing in 1996 in Dhahran, Saudi Arabia. The injuries and deaths in this bombing were almost exclusively due to flying debris.

Crowder. 2004

Figure 1.12
Khobar Towers Bombing, Dhahran, Saudi Arabia; Casualties from Debris, Not Collapse

1.7. What are the localized effects the members and connections must resist?

Studies[ii] have concluded that when a steel-framed structure is subjected to the shock of an air blast, or to severe localized impact, the following can occur:

- Twisting of beams—caused by lateral fixity of the beam's top flange created by direct attachment to the in-plane stiffness of the connecting floor diaphragm (assuming the floor diaphragm remains connected to beam flange), accompanied by lateral bending of those same beams framing into a common column. Such load phenomena are not typically accounted for in the design of a beam and its beam-to-column connections, and will enable gravity forces acting on contiguous floor systems to have an adverse effect on their supporting twisted beams and connections.

- Column effects—twist, local flange and web buckling, high dynamic shears at base and floor support.

1.8. How are blast loads different from seismic "loads"?

Blast loads are applied over a significantly shorter period of time (orders-of-magnitude shorter) than seismic loads. Thus, material strain rate effects become critical and must be accounted for in predicting connection performance for short duration loadings such as blast. Also, blast loads generally will be applied to a structure non-uniformly, i.e., there will be a variation of load amplitude across the face of the building, and dramatically reduced blast loads on the sides and rear of the building away from the blast. Figure 1.13 shows a general comparison between an acceleration record from a point 7 km from the 1994 Northridge epicenter and the predicted column loads for the 1995 Oklahoma City bombing.

It is apparent that the 12-second-long ground shaking from the Northridge event lasted approximately 1000 times longer than the 9 ms initial blast pulse from the Murrah Building blast.

The effects of blast loads are generally local, leading to locally severe damage or failure. Conversely, seismic "loads" are ground motions applied uniformly across the base or foundation of a structure. All components in the structure are subjected to the "shaking" associated with this motion. When a building is subjected to seismic loading, the distributed inelastic capacity of components and connections is available to mitigate global building failure; formation of a collapse mechanism requires damage (hinging) to occur in most of the beam-to-column connections in a given floor level. Figure 1.14 shows a traditional welded flange connection detail and its documented ductility under beam rotation (no axial tension load).

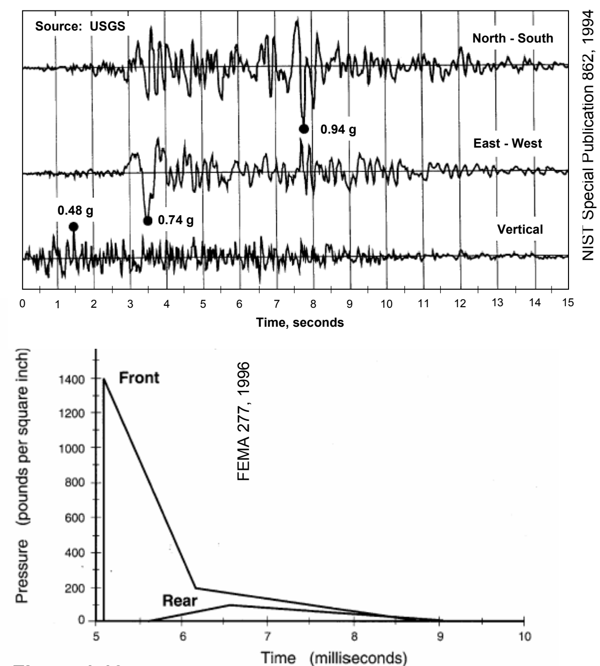


Figure 1.13
Comparison of Northridge Acceleration-Time History with Oklahoma City Bombing Column Load-Time History

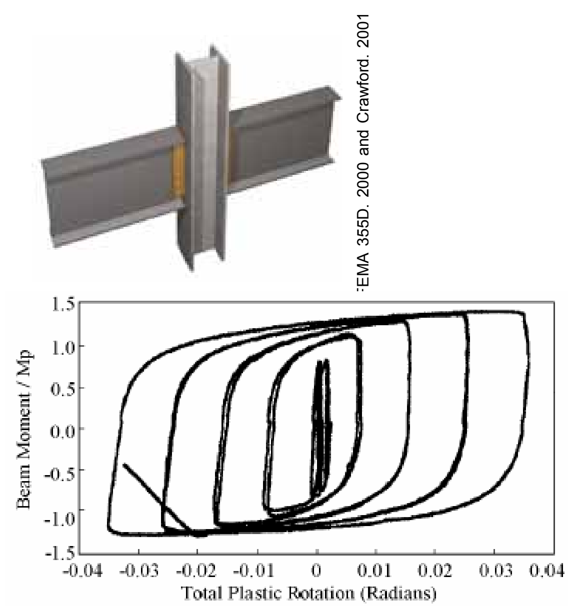


Figure 1.14
Traditional Welded Flange Moment Connection Schematic and Tested Performance

Conversely, for structures subjected to localized blast damage, failure of only a limited number of connections can be the mechanism for triggering progressive or disproportionate collapse.

Both blast loadings and seismic events are design issues related to life safety. However, significant but controlled damage is permitted.

Some aspects of seismic design can benefit structures also subjected to blast loads. While much of the connection research done after the Northridge earthquake has been beneficial in providing steel moment connection strong-axis rotational capacity information with some applicability to blast and progressive collapse designs, this research does not include many other factors critical to inelastic connection performance under the violent effects of blast loading. Some of these factors include severe member twist, lateral bending and material strain rate effects. Nor did this research account for those inelastic behaviors associated with progressive collapse mitigation, such as beam axial tension and moment interaction due to large displacements. Moment resisting frames with seismically qualified connections can help resist disproportionate or progressive collapse. However, as noted earlier, seismic ground motion is variable in amplitude and applied uniformly at the structure base, while blast loads are variable in amplitude, distribution and location. Thus, special features of seismic detailing that may work adequately in earthquakes, such as lateral stiffness provided by shear walls only, could be compromised in a blast event.

1.9. Are blast loads "service" loads or is blast resistance a "life safety" issue?

The design goal when considering blast loads in commercial and industrial building design is the reduction of casualties and the maintenance of the structural system in a "serviceable" condition only in as much as it permits egress after the extraordinary event. Special structures may require either partial or full containment (test cells, vapor containment areas, etc.) however.

Casualty mitigation and fatality prevention is achieved by preventing collapse and by limiting debris caused by the local failure of structural and non-structural elements. Debris reduction is achieved by proper detailing and the provision of ductility as previously discussed. Exterior walls, whether curtain-walls or infill systems, should be detailed such that brittle failure does not produce substantial debris. Windows can be designed to resist a nominal level of blast. These "blast-resistant" window designs will be detailed so that "punch" windows do not simply fail at the wall connections and become propelled into the interior spaces. In some cases, frangible elements such as windows can simply be replaced with non-frangible structures to eliminate the chance of injurious debris.

The General Services Administration (GSA) has guidelines for exterior windows for nominal blast levels that establish hazard levels associated with glass debris. Figure 1.15 presents these performance ratings for glass specified by GSA.

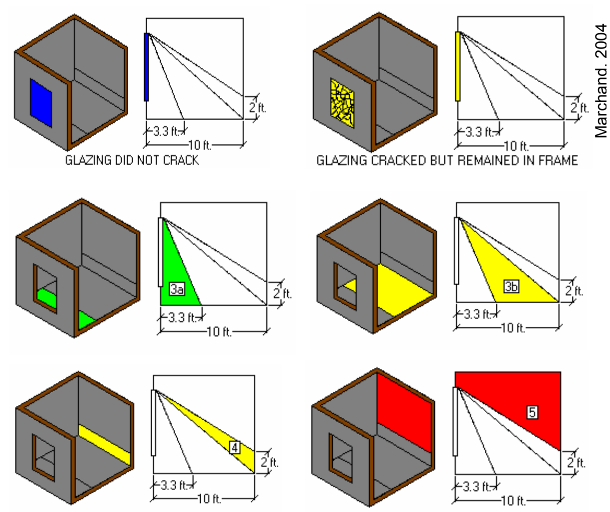


Figure 1.15
GSA Glass Debris Hazard Test and Analysis Criteria

Recent research[iii] has shown these criteria to be conservative, but they do facilitate specification of windows for blast resistance. The GSA/ISC (Interagency Security Committee) guidelines also specify minimum load levels for exterior walls and curtainwalls. Interior floors should be designed for uplift in the event of blast load propagation into spaces below.

Finally, progressive collapse criteria and design approaches described later in this document present strategies for limiting failure to local blast damaged areas such that disproportionate collapse doesn't occur.

1.10. What are the typical characteristics of a blast "wave"?

Blast waves can be characterized in terms of their peak pressure (amplitude), duration, shape and integral of pressure over time, or impulse. Blast waves from sources in unconfined spaces:

- generally have instantaneous rise times and exponential decay slopes, although complex geometries for external blasts and internal blast propagation can generate more complex waveforms.
- can be scaled, or characterized in terms of the scaled standoff Z from the source to the structure of interest.

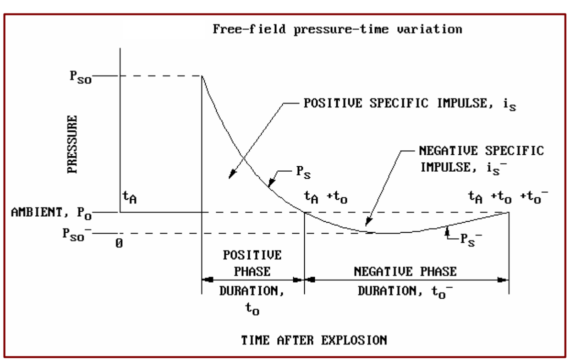
  The scaled standoff Z can be expressed as:

  $$Z = \frac{R}{W^{1/3}},$$

  where
  R = standoff distance
  W = explosive charge weight

- have both positive and negative phases (the negative phase results from the "recovery" of the atmosphere near the source of the explosion).

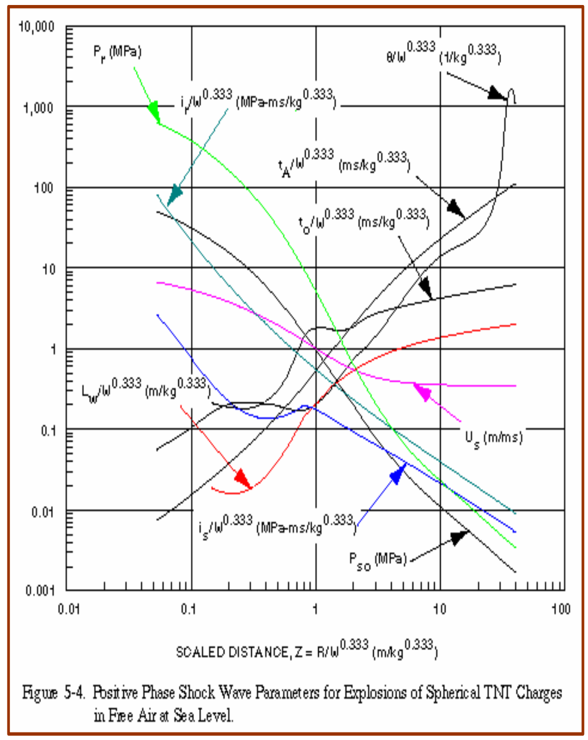
A simplified free-field blast wave is depicted in Figure 1.16.



Conrath. 2003

Figure 1.16
Free-Field Pressure-Time Variation

Both incident (unreflected) and reflected wave properties can be predicted using graphical tools. Figure 1.17 illustrates the graphical but somewhat complex blast prediction chart originally developed by the U.S. Army for the prediction of weapons effects. Scaled parameters for positive shock waves are presented. The horizontal axis of this chart is scaled standoff or Z, as previously defined.



Conrath. 2003

Figure 1.17
Positive Phase Shock Wave Parameters for TNT Explosions at Sea Level

The vertical axis defines values for three unscaled parameters:

reflected pressure (P_r),
side-on pressure (P_{so}), and
shock front velocity (U_s)

and numerous scaled parameters:

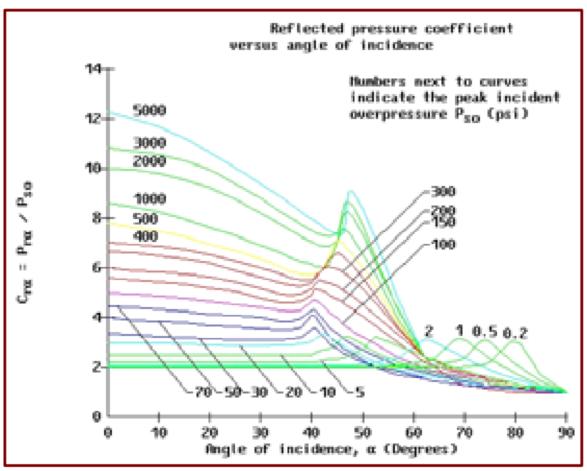
reflected impulse ($i_r/W^{0.333}$),
side-on impulse ($i_s/W^{0.333}$),
time of arrival ($t_A/W^{0.333}$),
positive phase duration ($t_o/W^{0.333}$),
positive phase wavelength, ($L_w/W^{0.333}$), and
time constant, ($\theta/W^{0.333}$)

where W is the explosive charge weight (in this case in kg).

Similar charts are available for free air explosions and for negative phase parameters of both surface bursts and free air explosions.

The fully reflected values presented in these curves do not include the effects of clearing associated with reflecting surfaces of limited size. Methods are available to calculate reduced impulse associated with clearing in technical manuals[iv] and in software such as CONWEP.

Explosions in confined spaces or where partial confinement exists (urban "canyon" settings where adjacent buildings provide partial confinement) have more complex wave shapes and parameters. The reflecting surfaces in these scenarios generate amplifications of pressure and the resulting impulse. Figure 1.18 shows a predictive curve for blast wave reflection factor as a function of peak incident pressure and angle of incidence of the wave-structure interaction.



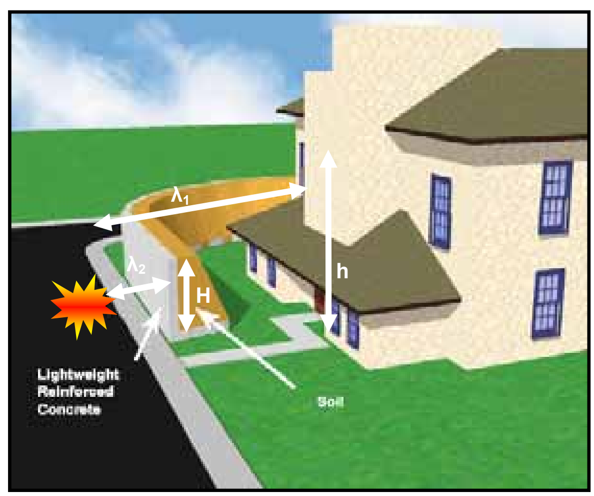
Conrath. 2003

Figure 1.18
Reflected Pressure vs. Angle of Incidence

1.11. Can't we just shield our building from the blast?

So-called "blast walls" can be constructed to shield a building or structure from the blast waves by providing the reflecting surface for the blast, and limiting the load on the structure of interest to some fraction of the predicted reflected load. Unfortunately, the effectiveness of such walls is highly dependant on the height of the wall and the distance from the rear of the protective wall to the structure or building. The protective wall would be required to resist the full reflected blast load. Thus, if overloaded, the wall could be a source of significant hazardous debris.

An example of a possible configuration is shown in Figure 1.19. Here a blast wall about 10-ft tall is placed to provide protection for a structure about three times its height. The wall is backed with soil to mitigate the effects of the concrete debris should the wall fail. A predictive curve for impulse reduction from TM 5-853 is shown in Figure 1.20. When the factors are unscaled using a 1000-lb charge in the street, an "adjustment factor" for impulse can be calculated for the geometry defined in Figures 1.19 and 1.20. In this case, impulse in the middle of the wall is reduced to about 70 percent of its original value without the blast wall. It should be noted that more recent research[v] has shown that better reductions can be achieved, depending on geometry. That same research has shown that adjustment factors can be greater than 1.0 (worse than without blast wall) for some geometries as well.



Humphreys, 2001

Figure 1.19
Schematic of Blast Wall and Parameters

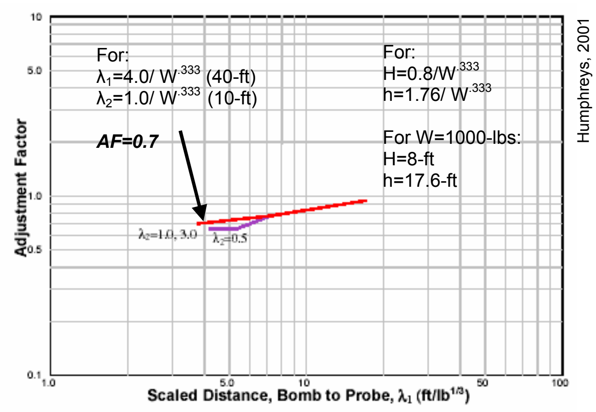


Figure 1.20
Adjustment Factor Calculation for Impulse for Blast Wall

1.12. Can blast loads be applied as uniformly distributed loads?

Where the angle of incidence between the explosive charge and the structure or structural element edge or end exceeds 45 degrees, significantly reduced pressures and impulses are applied to the structure. So, when the charge standoff exceeds one-half of the structure or element width or height (assuming the charge is centered on the structure or element) loads can be reasonably accurately averaged over the structure or element. For shorter standoffs, the loads should generally be described as a non-uniform distribution.

Figure 1.21 illustrates the possible variation in load over a surface due to geometry.

Three simulated 12-ft-tall, 12-in-wide blast loaded columns are shown; one with a 125-lb charge at 5 ft, one with a 1000-lb charge at 10 ft and one with a 27,000-lb charge at 30 ft. Blast scaling suggests that peak pressures should be essentially identical at these standoff-charge weight combinations, but geometries (angles of incidence) dramatically affect the distribution of pressure over the height of the columns.

Figure 1.22 illustrates the complex blast environment generated in an urban setting. Multiple buildings create many reflecting surfaces for blast waves to encounter.

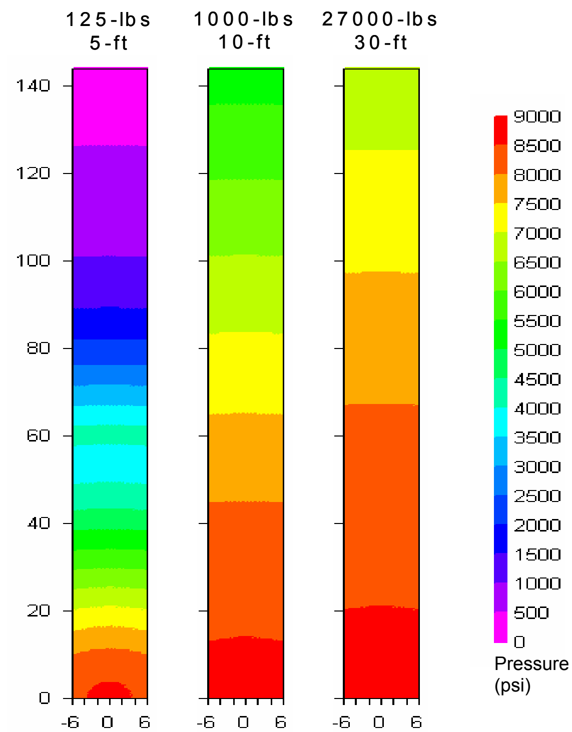


Figure 1.21
Comparison of Pressure Distribution for Columns loaded with Identical Peak Pressure at Different Scaled Standoffs

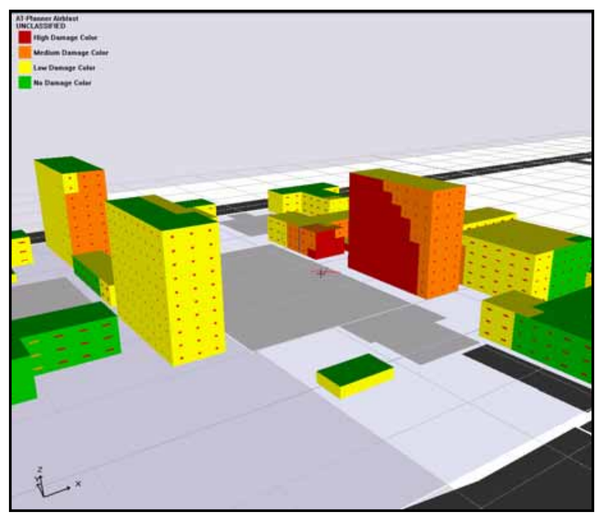


Figure 1.22
Simulation of Pressure Distribution in an Urban Environment

1.13. Isn't there an "equivalent static" design approach available?

Generally, no. The capacity of a structural system to resist blast is a function of both its material linear (elastic) and material non-linear (inelastic) capacity and its inertial resistance (mass). Attempting to design to peak blast pressures will likely prove to be impossible or at least highly inefficient.

Simplified methods can be used, however, to "scope" or to gain a "feel" for the performance of a blast-resistant design. Work and energy solutions bounding the extents of inelastic response have been developed for a variety of structural components. So-called "pressure-impulse" or *P-i* diagrams graphically present asymptotic limits for cases where the change in peak pressure changes insignificantly over the time to maximum response (pressure dependant) and for cases where the blast load duration is short with respect to the time to maximum response (or natural period of the structure.) An example of a *P-i* diagram is shown in Figure 1.23.

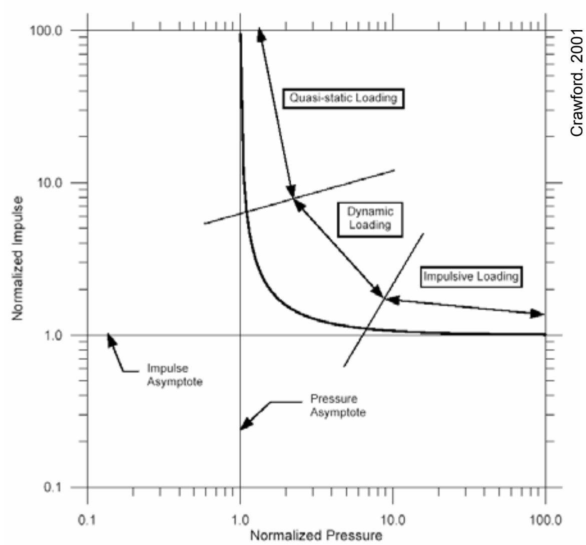


Figure 1.23
***P-i* Diagram Parameters**

1.14. What tools are available to calculate blast loads?

The complete set of blast prediction curves are available in public documents such as TM5-1300, "Structures to Resist the Effects of Accidental Explosions", and in Government contractor restricted documents such as the DAHS manual (UFC 3-340-01, Design and Analysis of Hardened Structures to Conventional Weapons Effects). Algorithms have been developed to generate the parameters illustrated on the graphical blast prediction tools described above. These polynomial fit-based equations, as well as equations that describe and account for angle of incidence and reflection coefficients, are incorporated into software programs such as ATBlast (publicly available from GSA) and CONWEP (restricted to Government contractors, but available from the USACE ERDC in Vicksburg, MS).

More sophisticated programs are available to account for multiple reflections in complicated geometries, variable explosive formulations, varying environmental conditions as well as multiple explosions. One of these tools is BlastX, a restricted-use software tool jointly supported by the USACE and USAF.

Finally, special purpose finite element and finite difference codes such as LSDYNA and ABAQUS and computational fluid dynamics codes (CFD) are becoming more practical for use in these types of problems because of improved algorithmic and numerical efficiencies and computer hardware capabilities.

1.15. What about other extreme loads like vehicle or aircraft impact? Can those loads be quantified?

Air blast loads are generally considered uncoupled with the structure. In other words, the response of the structure does not "feed back" into the load pulse, changing its fundamental properties. Impact loads like vehicle or aircraft strikes on structures are complex problems where impactor (vehicle or airplane) stiffness and crushing strength determine the loads imparted to the structure. Special purpose finite element analysis tools are required to evaluate these scenarios in detail. Some simplified methods are available for use in designing simple vehicle barriers, however. One such tool incorporating these methods is the BIRM3D software developed and maintained by the USACE, Omaha Protective Design Center. A BIRM3D simulation is shown in Figure 1.24.

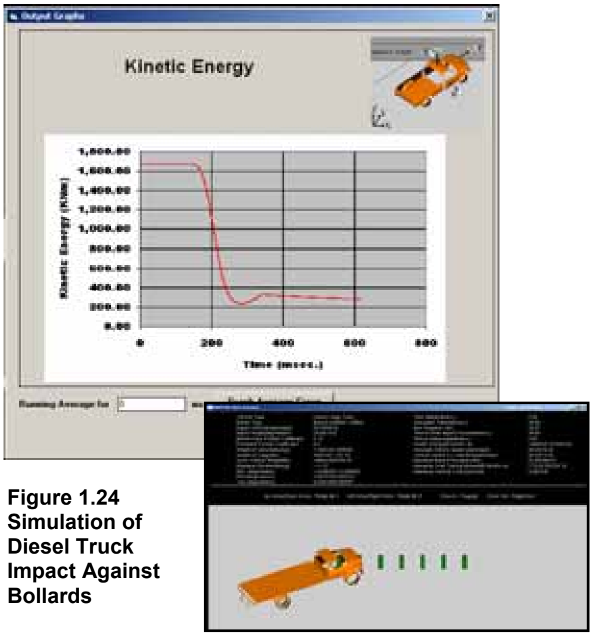


Figure 1.24 Simulation of Diesel Truck Impact Against Bollards

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[i] "Minimum Design Loads for Buildings and Other Structures," SEI/ASCE 7-02, Section C2.5: Load Combinations for Extraordinary Events, 2003, page 242.

[ii] Crawford, J.E., et al, "Design Studies Related to the Vulnerability of Office Buildings to Progressive Collapse Due to Terrorist Attack," Karagozian and Case TR-01-10.1, October 2001.

[iii] Marchand, K.A., et al, "Blast Induced Glass Hazards: Comparison of Design Approaches and Recent Research; Is it Time for Improved Hazard Models?," presented at the ASTM F12 Symposium on Building Security in an Age of Terrorism, October 2004.

[iv] "Design and Analysis of Hardened Structures to Conventional Weapons Effects," Unified Facilities Criteria (UFC) 3-340-01, "Department of Defense, for official use only, June 2002, Loads on Structures, Section 9.3.3, page 9-12.

[v] Humphreys, Edward A. and Piepenburg, Dwayne D., "Assessing Effectiveness of Blast Walls: Final Report for the Period 1 January 2000 to 31 December 2000," SAIC-01/1003, prepared by Science Applications International Corporation, prepared for Combating Terrorism Technical Support Office, January 2001.

SECTION 2 DETERMINING THREATS AND ACCEPTABLE RISK

2.1. Shouldn't level of risk determine the extent to which I design for these loads?

Yes. Existing criteria for structures required to be designed to resist blast, impact and progressive collapse include approaches to quantifying risk and corresponding structural performance criteria. In general, buildings under the purview of the General Services Administration (GSA) and agencies that have adopted the GSA/Interagency Security Committee (ISC) criteria and buildings designed in accordance with DoD (Department of Defense) Unified Facilities Criteria (UFC) will have progressive collapse mitigation requirements as a minimum and may have prescribed blast threats as well.

Design or evaluation of buildings where GSA or DoD requirements do not apply can be supported with threat assessments where the likelihood of blast and impact threats are quantified. Security professionals with access to local and national threat information who are also familiar with building vulnerabilities should be engaged to provide this information. This specialist will evaluate vehicle and personnel access to the facility, receiving/mail/supply delivery activities, line-of-sight opportunities for ballistic attack, and existing operational security procedures to determine and quantify explosive, impact and ballistic threats.

The TM 5-853 Manual Series (TM 5-853-1/2/3), currently being updated (scheduled to be released as UFC 04-020-01/02) provides detailed descriptions of approaches for threat and risk assessment. Other UFCs are scheduled to address vehicle barriers (UFC 04-020-02), fences, gates, barriers and guard facilities (UFC 04-020-03) and entry control points (04-020-01).

A threat and risk assessment will address the following issues and steps as a minimum:

1. Definition of Assets:
   - Valuation of assets to be protected
     - Critical to operation?
     - Hard or soft (hardware or information)?
     - People?

2. Definition of Aggressors
   - Type of facility to be protected
     - High visibility?
     - National or local historical, political or esthetic value?
     - Accessible?
     - Recognizable?
     - Value to the aggressor ("bang for the buck")
     - History of attacks/success?
     - Law enforcement visibility/deterrence?

3. Definition of Tactics and Threat Severity Levels
   - Match tactics with
     - Aggressors
     - Assets
   - Determine threat severity level (tools and tactics)

4. Combine above into design basis threat

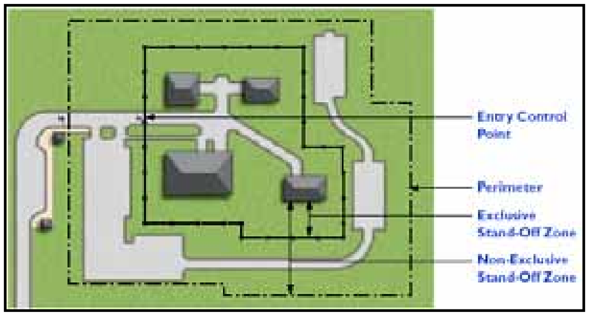
5. Determine level of protection
   - Related to asset value and design basis threat

6. Determine level of risk and acceptability of risk
   - Related to certainty associated with tactics and protection

7. Identify user constraints
   - Cost
   - Existing operational controls
   - Adjacencies/location
   - Architectural/esthetic constraints

Building and site design should be approached holistically considering operational security, maximized standoff, and vehicle counter-mobility. Protection must be considered in the functional layout with special consideration of high threat and vulnerable adjacencies.

An example of site security considerations (site perimeter protection concepts), is shown in Figure 2.1.



USAF Installation Force Protection Guide

Figure 2.1
Site Perimeter Protection Concepts

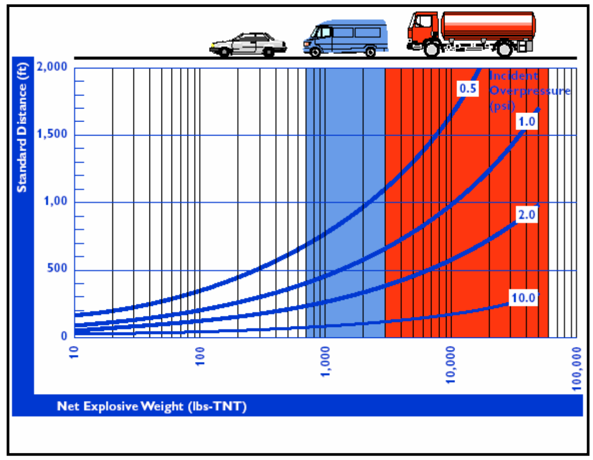
2.2. How is the likelihood of a deliberate attack involving blast determined?

Two types of deliberate attacks using blast loads are generally considered; vehicle bomb attack and person-carried explosive attack. Vehicle bomb attacks historically have ranged in size from tens of pounds of explosives to the approximately 20,000 lbs of material detonated near the Khobar Tower troop dormitories in Dhahran, Saudi Arabia. Figure 2.2 shows explosive cargo range and incident overpressure associated with different vehicle bombs and standoffs.

Vehicle bomb attacks are limited by vehicle access and approach points. Enforced standoff between the nearest desired approach of unrestricted vehicles must be provided; terrorists don't obey traffic signs and don't mind bumping over curbs and lawns. Vehicle barrier designs and products are available to make perimeters inviolate.

Person-carried explosive devices are generally considered to be less than 100 lbs, and are usually assumed to be approximately 50 lbs.

The likelihood of either of these attacks taking place and the definition of the area over which the loads are applied is a function of accessibility and closest proximity of the threat explosive. Buildings may also be threatened simply because of their proximity to targets of higher value. Major bombing events such as the Murrah Building bombing in Oklahoma City have caused significant "collateral damage" in nearby buildings.



USAF Installation Force Protection Guide

Figure 2.2
Vehicle Bomb Sizes, Standoffs and Overpressures

In any case, maximizing Stand-off distance will decrease the effects of a blast from a particular charge weight because the blast pressures are reduced significantly as distance increases. While maintaining adequate stand-off is an effective means of reducing the risks from blast effects, it alone may not be enough. Satchel charges, weapons effects and uncertainty of defined threats may subject a structure to unforeseen levels of blast loadings. Therefore, some measure of prescriptive protection, such as designing for notional member removal, detailing members and connections to ensure ductile performance, providing continuity and redundancy both globally and locally, should be incorporated into the steel frame design.

2.3. Does the structure have to do all of the work? What about operational and other "non-structural" security measures?

Protection of buildings and building complexes should be thought of in terms of "layered protection", "rings of protection" or "defense in depth". The desired goal is to identify and mitigate threats at the outermost ring of security possible. All threats do not become apparent at the same layer or ring of security, therefore, the use of a layered and integrated security approach is critical. Figure 2.3 shows a "layered" concept for a bank building.

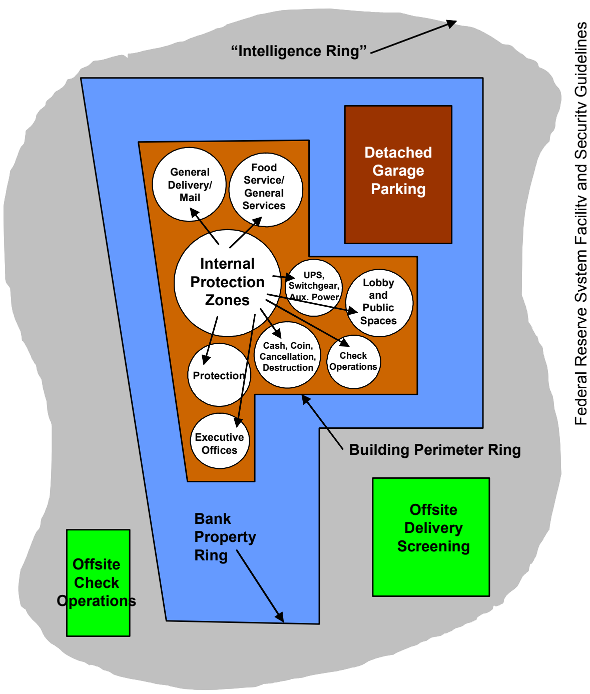


Figure 2.3
Example of "Layers" or "Rings" of Protection

Effective layered protection involves the use of all protection resources, including:

- ongoing liaison and timely communication with local and national sources of threat and intelligence information;
- site surveillance;
- effective use of site property, terrain and interface with public access and thoroughfares;
- effective use of administrative control and vehicle and personnel identification;
- consistent use of vehicle and personnel inspection and screening technologies coupled with effective protocols for their use;
- incorporation of physical barriers for vehicles and personnel;
- use of state-of-the-art access control hardware and technologies;
- incorporation of protection features into building component design and construction; and
- effective use of protection staff in conjunction with protective measures.

As implied in the threat and risk definitions above, operational constraints and controls are an important part of the overall site security and protection scheme for a building and site. Perimeter vehicle barriers can enforce standoffs for blast. Figure 2.4 shows perimeter protection concepts that are not architecturally "offensive", developed for the National Capitol Planning Commission (NCPC) in Washington, DC.

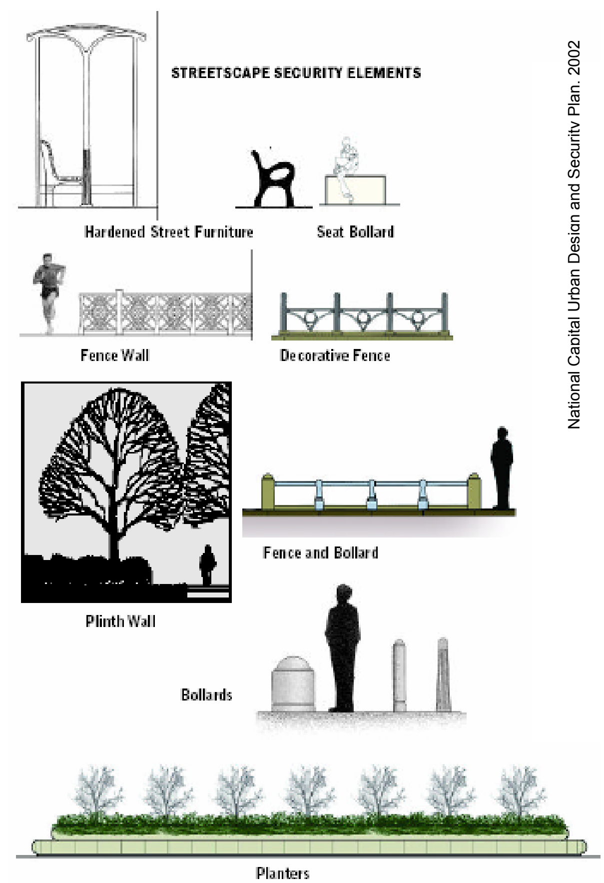


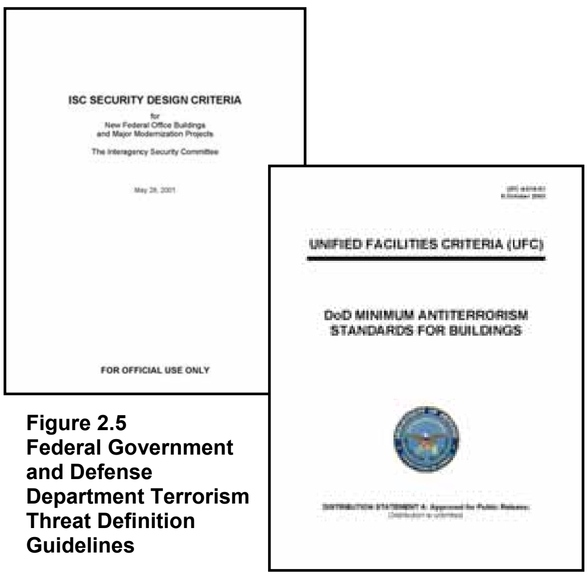
Figure 2.4
Architecturally Contextual "Streetscape" Vehicle Barriers

Access controls at building entrances can restrict person-carried explosives and eliminate the necessity for blast hardened design of internal structural components. Building deliveries, mail and packages can be inspected offsite or in a detached facility, thereby limiting exposure to blast in a main building.

In addition to site use and restrictions, internal space allocation can be designed to limit human exposure to blast effects and debris. Perimeter spaces with minimal standoff can be reallocated for storage or restricted from regular occupancy.

2.4. What do current guidelines require of the structure?

In terms of threat, risk and corresponding blast loads or level of protection requirements for progressive collapse assessment, standard design guides such as ASCE 7-02 and current building code provisions (UBC and IBC) provide no specific guidance for determination of threat.


ISC SECURITY DESIGN CRITERIA
for
New Federal Office Buildings
and Major Modernization Projects
The Interagency Security Committee
May 28, 2001
FOR OFFICIAL USE ONLY

UNIFIED FACILITIES CRITERIA (UFC)
DoD MINIMUM ANTITERRORISM STANDARDS FOR BUILDINGS

Figure 2.5
Federal Government and Defense Department Terrorism Threat Definition Guidelines

The GSA/ISC criteria (Figure 2.5), parts of which are restricted to GSA contractors, do address threat and risk by prescriptively assigning required standoffs and charge weights and specific blast loads (pressure and impulse) for the building envelope based on levels of protection assigned to specific facilities. Progressive collapse analysis is required for nearly all GSA facilities. DoD facilities require a threat assessment for every design. This assessment may simply defer to a prescriptive set of minimum protective measures presented in UFC 4-010-01, "DoD Minimum Antiterrorism Standards for Buildings" and UFC 4-010-10, "DoD Minimum Antiterrorism Standoff Distances for Buildings" (restricted). (Figure 2.4) The procedures contained in UFC 4-011-01, "Security Engineering Planning Manual," will define specific threats for DoD facilities beyond the minimum standoff requirements. A number of prescriptive guidelines also exist for a variety of other Government agencies.

For non-federal projects and commercial buildings, an independent threat assessment may be conducted as required by the developer or owner.

2.5. How do I quantify damage as it relates to loads corresponding to a specific level of protection?

Acceptable structural performance for blast, impact and progressive collapse is tied to hazard mitigation and casualty prevention. Generally, since inelastic behavior is permitted in elements subjected to these loads, performance is deformation controlled; i.e., some allowable rotation or ductility (based on minimizing debris) is specified for each structural member type for blast loads. For progressive collapse, deformation limits are specified to ensure some residual capacity exists. Presently, recently published deformation criteria or "response limits" for blast loads are restricted as contained in the UFC 3-340-01 DAHS manual. Progressive collapse criteria are presently unrestricted and contained in the GSA Progressive Collapse Guidelines and in UFC 4-023-03.

2.6. Is there a significant increase in cost associated with the determined risk?

Structural design improvements to mitigate the effects of blast and progressive collapse generally may not add more than 5% to the design and construction cost of a new facility, regardless of the type materials used (i.e., concrete, steel, masonry, etc.)[vi] This percentage could be significantly higher should a building or facility be deemed "critical" where special hardening measures are employed.

For existing facilities, improvements will be substantially more expensive, and often are not even considered until scheduled major renovations to the facility are contemplated. Often, comprehensive upgrade costs will approach the total replacement cost of the facility. Thus, costs for partial improvements must be weighed against the potential loss (risk). GSA has developed tools to assist its project managers in developing budget level cost estimates for the impact of security. A computer program GSACOS (GSA Cost of Security) was developed to account for the major security items included in a construction project (typically a courthouse). Information about this computer tool and how to access it are available on the software portions of the GSA's Office of the Chief Architect Technology Transfer website (www.oca.gsa.gov).

2.7. Who is responsible for determining the required protection?

Even when prescriptive requirements are satisfied by the designer, some risk associated with the uncertainty of threats and tactics remains. The owner, developer or owner-occupant must make the final decision on level of protection and risk. As mentioned previously, qualified security professionals can be engaged to support these decisions.

2.8. What are the legal liabilities associated with the selected design approach?

Because prescriptive and performance requirements for blast and progressive collapse mitigation do not appear in the design codes and, by reference, the building codes (IBC and UBC), they are not codified into law. The legal requirements for the designer are the specific terms of his or her contract. Blast and related security design is a specialized field requiring significant experience and expertise. Like other special project requirements, sufficient expertise should be present on the design team, either provided by the primary structural engineer or by special consultants.

[vi] "Protecting Buildings from Bomb Damage: Transfer of Blast-Effects Mitigation Technologies from Military to Civilian Applications, " Commission on Engineering and Technical Systems, National Research Council, National Academy Press, 1995, Appendix A, Financial Performance of a Commercial Office Building, Table A-2, page 84.

SECTION 3
RESISTANCE OF STEEL STRUCTURAL SYSTEMS TO BLAST AND LOCALLY EXTREME LOADS

3.1. How do blast mitigation and collapse mitigation differ?

Blast mitigation or blast-resistant design of a steel structure involves the evaluation of individual structural members or groups of structural members, depending on the distribution of blast load. The design intent is to reduce or eliminate debris, thereby providing a protective "envelope" for assets or people inside the building, or to prevent failure of a key structural element, thereby providing specific local resistance to the blast load.

Conversely, design to resist progressive collapse is, in a sense, threat independent, in that the design intent is to provide sufficient redundancy in the structural system to prevent the propagation of failure. Local failures are assumed to occur, and in the most widely accepted approach, a single vertical structural component (typically a column) is removed, and the structure required to "bridge" over that removed component. Much of progressive collapse design, then, concerns a "distribution" of redundancy throughout the structure such that bridging can occur wherever a column may be lost.

3.2. Are "efficient and readily erectable" structures and "blast-resistant" structures mutually exclusive terms?

Any blast-resistant structure must be designed to provide the capacity required to resist applied blast loads both locally at the component and connection level and globally at the structural system level. In other words, a robust, redundant and ductile structure will perform best against the severe transient loads produced by an explosion. Some have suggested that reinforced concrete is a superior material for blast resistance due to its massive and monolithic nature. Properly detailed monolithic reinforced concrete structures with inherent mass do provide good performance. Steel structures, properly detailed with member and connection ductility, also perform well. Concrete structures with discontinuities and instabilities in certain load directions (precast/prestressed beam, joist and floor systems) can perform very poorly under blast loads.

Steel structures have suffered more from a lack of basic design information related to blast loading than they have suffered the direct physical effects of blast waves over the years. One regrettable legacy of the Cold War nuclear weapons testing program is the perception that reinforced concrete is the "material of choice" for blast-resistant structures. Some recent literature has promulgated these somewhat less than supportive statements concerning structural steel:

> "Generally, well designed cast-in-place reinforced concrete structures provide a significant level of protection against explosive loads compared with other materials. This is because of their heavy weight, monolithic character, and ductile response. Steel frame construction may also be effective in resisting explosive effects. For steel structures, the frame has considerable ductility if the connections are properly designed and constructed. However, the materials used for the floor system, the exterior wall cladding, and their connections to the steel frame often have less resistance to explosive effects. Consequently, the frame may survive, but the occupants and contents may not."[vii]

The fact is that proper detailing of fenestration, cladding and floors, redundant gravity and lateral load resisting systems and generally ductile structural system response determines the efficacy of any blast-resistant structural design. When incorporated into a building design, these characteristics, coupled with both fabrication and erection efficiencies, make structural steel equally as good and possibly a better choice for blast resistance.

3.3. Aren't all blast-resistant structures heavy, ugly and expensive?

No. Ductility and redundancy don't necessarily mean that a heavy or "bunker-like" appearance is required. The Pacific Command Center recently designed for the Department of the Navy's Pacific Division by Baldridge & Associates Structural Engineering, Inc., Honolulu, HI, (Figure 3.1) is an example of an attractive and efficient design incorporating aspects of blast resistance and progressive collapse mitigation required by the DoD guidelines.

Baldridge, 2003.

Figure 3.1
Pacific Command Center and Proprietary Blast-Resistant Connection Detail

3.4. What is "Balanced Design"

Design to resist blast or other extreme loads must be balanced in the sense that hardening of one key component will likely transfer loads to adjacent supporting components that may or may not be designed to resist these additional loads. An example is a cladding or curtain wall glazing system that is upgraded to resist blast or at least to provide resistance to higher blast loads. This curtain wall will now transfer more of the blast load (more of the pressure for a longer duration) to the floor and column framing (lateral system) than the non-hardened system. These loads must be taken into account to prevent a "weak link" from compromising the structural system.

3.5. What types of steel structural systems work best to resist blast and other extreme local loads?

Ductile moment resisting frame systems (i.e., SMF as defined in AISC 341-02) with a frame layout designed to maximize redundancy will generally perform best.

Connections designed and detailed as described in the recommendations in Section 4 should improve blast resistance when used in a well-designed global structural system.

3.6. What are some general design geometry strategies to suggest to the architect to mitigate blast effects?

The shape of a building can have a contributing effect on overall damage to the structure. For new buildings, a regular and uniform layout of structural elements (beams, columns, walls, etc.) can have a significant impact on the ability of the structure to withstand blast-induced progressive collapse. Regularity in design allows for continuity of strength, greater redundancy and easier redistribution of load should an element fail. Irregularities, such as reentrant corners and overhangs, can trap the shock wave and amplify the blast loads. In general, convex, rather than concave shapes are preferred for the exterior of the building. Reflected pressures will be reduced on a convex surface as compared with flat or concave surfaces.

Some other general recommendations include:

- Limit column spacing. Large column spacing reduces the ability of the structure to span over lost vertical elements. Note, however, that small column separation can increase the vulnerability of two adjacent columns depending on explosive charge size.
- Exterior bays are the most vulnerable. Shallow exterior bays should be used if possible.
- Transfer girders or columns supporting transfer girders can contribute significantly to overall building damage.

Add redundant transfer systems when transfers are required on the building exterior.

- Architectural treatment of exposed exterior columns can dramatically improve survivability of those columns. Every inch of additional standoff from hand placed or person-carried explosive charges to columns will significantly improve survivability.
- Closely spaced beams framing into girders may improve load redistribution upon failure of a single beam.
- Consider cantilevered first bays from a recessed first column line to the building perimeter. This creates additional standoff for the first column line. Cantilevered bays must be designed for uplift, however.
- Lighter wall cladding will reduce transferred loads into the structural system.
- Allow cladding to be attached to and span vertically between floor slabs.

3.7. Are there some general relationships between explosive charge weight, distance and damage?

Yes. Simple charts and graphs exist to characterize building and component damage generated by blast loads.

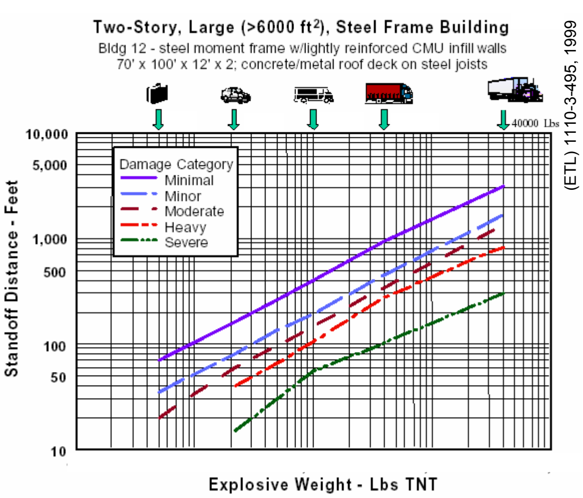


Figure 3.2
Simplified Characterization of Damage for Steel Industrial Building Subjected to Blast Loads

DoD vulnerability assessment tools such as ATPlanner and BEEM make use of these simple approaches. Figure 3.2 shows one such chart for a steel industrial building.

These tools are beneficial in providing a "feel" for blast capacity of structural systems and components, but should not be used for design.

3.8. Is strength alone the answer? What role does weight (mass) play? Are particular material or geometric section properties required?

Ductility is the right answer. Ductility is the ability of a material or assemblage to undergo an appreciable amount of permanent deformation, including plastic strain of the material, prior to rupture or prior to approaching some other limit state of the assembly. Strength and stiffness are important, and mass of the component directly loaded by the blast will contribute to overall resistance by adding inertial resistance. Mass also implies thickness, which, for steel members and connections, can help by providing local shear and tearing resistance and resistance to local buckling.

Existing design guides such as UFC 3-340-01 recommend the exclusive use of so-called "seismic" sections (per AISC 341-02).[viii] This is not a requirement.[ix,x] The allowable member deformations given in UFC 3-340-01 are large (increased flexural ductility is assumed) based upon the assumption that "seismic" sections are used. "Seismic" sections are more likely able to achieve the expected level of performance without premature buckling for these sections.

Figure 3.3 shows a ductile seismic connection that might be used in blast-resistant design.

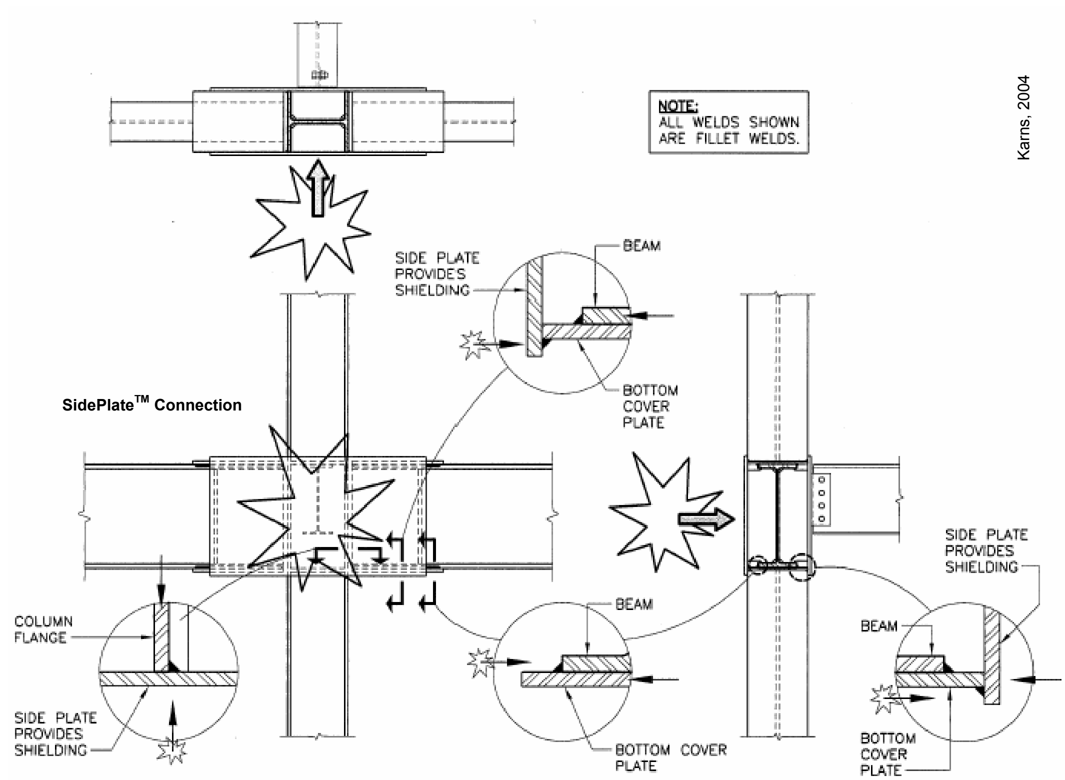


Figure 3.3
Example of Ductile "Seismic" Steel Connections Also Providing Blast Protection

3.9. How important is connection design?

For steel structures, the performance of the structural system is only as effective as the connecting elements. Connections are the most vulnerable elements within the system. Proper design and detailing of steel connections is essential to achieving ductile and robust performance under blast loads. Steel components, when properly detailed, can have the necessary ductility to provide good resistance to blast and progressive collapse effects. Because of the perceived difficulties in creating a congruent, "homogeneous", structural system from a series of discrete inter-connecting components, many blast designers have shied away from structural steel.

Thus, connections are almost exclusively the issue that has raised the most questions in the blast-resistant design of structures, given the vast array of connection types, configurations, and the varying degree of ductility found in each. From the earliest guides to the present, minimal information has been available related to dynamic inelastic steel connection design (TM 5-1300, the seminal reference in the field of blast-resistant design, contains 197 pages related to steel member design, with only a scant 4 pages dedicated to connections. TM 5-1300 does, however, attempt to alert the reader to the need of ductile connection detailing, stating "Special care must be taken in steel design to provide for connection integrity up to the point of maximum response.").

Limited research has been completed to date to determine the performance of steel connections under blast loading. Recent work in the seismic research field has contributed much needed information regarding the performance of fully rigid moment connections under large static loads and strong-axis rotations. The SAC tests and subsequent report series, particularly FEMA 355D, "State of the Art Report on Connection Performance" and more recent AISC guidance

("Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications") (Figure 3.4) provide detailed and timely guidance for seismic design applications (i.e., cyclic semi-static loads) that is applicable to connections requiring substantial ductility and rotational capacity.

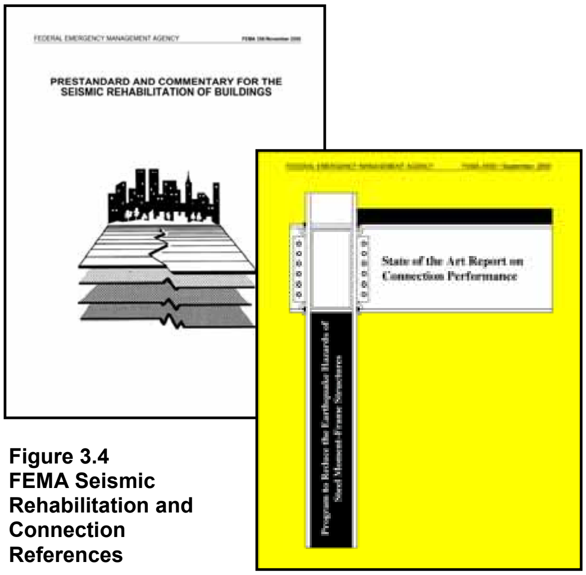


Figure 3.4
FEMA Seismic Rehabilitation and Connection References

This series of tests and detailed reports provides much useful information, such as the performance of a weak partially restrained connection (bolted shear connection) shown in Figure 3.5, and non-ductile behaviors of typical connections (Figure 3.6).

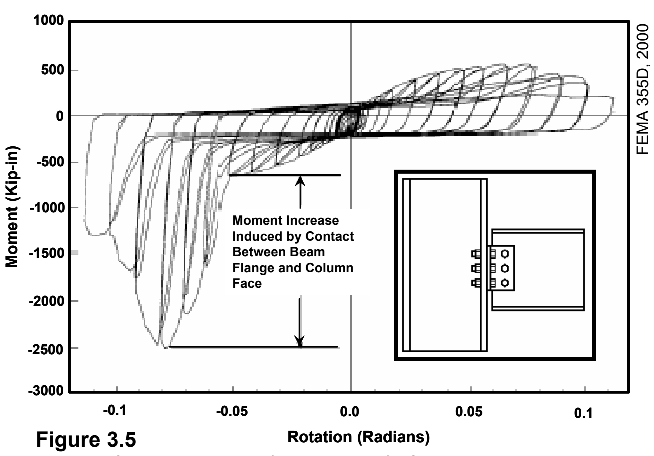


Figure 3.5
Plastic Connection Performance of "Simple" Beam-to-Column Connection Illustrating Surprising Connection Ductility Under Semi-Static Loading

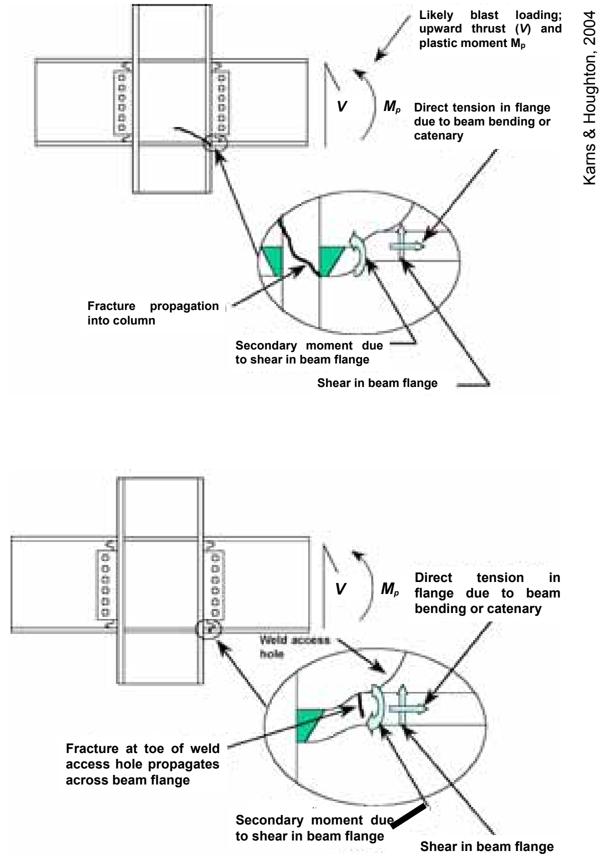


Figure 3.6
FEMA Connection Research Identifies Nonductile Behaviors due to Geometry-Induced Stress Concentrations

Little data exists to quantify the performance of these steel connections directly loaded by blast. However, seismic tests of common connection geometries can provide limiting rotations for connections supporting blast loaded components.

This testing has also highlighted that steel frame connection geometries that introduce stress concentrations are inherently non-ductile and lack robustness even under statically applied load. While SAC/FEMA research has direct design utility in the determination of strong-axis rotational capacity of moment connections, the testing that forms the basis for the guidance provided does not include those factors important to inelastic connection performance under the violent effects of blast loading such as severe member twist, lateral bending and material strain rate effects. SAC/FEMA testing also did not account for those inelastic behaviors associated with progressive collapse mitigation, such as beam axial tension and moment interaction due to large displacements. Several concurrent research projects are in progress sponsored by DTRA and GSA to investigate the effects of these behaviors on connection performance. Until the results of this blast research are made available to design engineers, steel frame connection design should include the following:

- Essential connection attributes identified in Section 4.6 of this document should be incorporated
- Connections should meet the rotational capacity provisions of AISC 341-02 for special moment frames
- Design connections for two limit states:
  1) beam plastic moment and
  2) beam axial tension capacity
  These limit states must also be checked for deformation compatibility
- Use moment connections for beams in both directions from the perimeter, i.e., allow beams to cantilever from one bay in from the exterior
- Size bolted connections such that failure modes with lesser energy dissipation mechanisms (e.g., block shear) do not control
- Size shear-only connections to be capable of developing sufficient beam axial tension
- Consider high-strength bolted connections to prevent brittle failure from concentrated stress at weld locations
- Use weld metals with inherent notch toughness. The requirements of AISC 341-02 for weld metal and quality assurance should be followed.
- Avoid connections that tend to concentrate ductility in small regions of the structure. Unlike the case in Seismic design, it is preferable to distribute ductility along members, so as to minimize concentrated strain demands.

3.10. How does cladding and glazing contribute to lateral resisting system loads?

Cladding systems and glazing collect at least a portion of the total blast load and transfer that load to supporting spandrels, beams and columns through cladding system and glazing reactions. Load is transferred to the supporting members only as long as the cladding system and glazing offers resistance to the blast load. After failure, no additional load is transferred. In general, cladding systems responding to blast loads should be designed in balance with the supporting frame or structure. Overly stiff or strong cladding may actually be detrimental to the overall structural system response. Note that even the lightest cladding systems will survive long enough under blast loads from conventional explosives to allow full reflected pressures to develop, however.

3.11. How does floor response contribute, especially for internal explosions?

Floor systems, particularly those located on perimeter bays of the structure, can be loaded by blast that infiltrates interior spaces through failing cladding or glazing. At best, these floor systems will respond to the blast and reaction loads will be transferred into supported beams or girders. At worst, these floor or floor and joist systems will fail, particularly if not provided with design capacity for uplift, and will leave supporting beams, girders or columns without lateral support, thereby increasing the unsupported lengths of those members and making them susceptible to stability related failures. It is also doubtful that ordinary shear studs could always provide sufficient slab-to-beam anchorage in floor systems subjected to uplifting blast loads.

3.12. What are some designs and details to avoid?

Some obvious systems and details to avoid include:

- Connection details controlled by non-ductile or brittle failure modes
- Non-redundant transfer systems on the building perimeter
- Widely spaced columns or lateral elements
- Façade or cladding geometries that "collect" blast load due to their concave shape or re-entrant corners
- Strong cladding/weak framing scenarios where cladding failure will not occur prior to frame failure
- Weak column splices near lower levels

[vii] "Lessons From the Oklahoma City Bombing – Defensive Design Techniques," Chapter 19, "Structural Countermeasures", pages 33-34, ASCE, 1997.

[viii] "Design and Analysis of Hardened Structures to Conventional Weapons Effects," Unified Facilities Criteria (UFC) 3-340-01, "Department of Defense, for official use only, June 2002, Loads on Structures, Section 10.8.3, Mechanics of Structural Elements, Beams, page 10-48.

[ix] "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects," US General Services Administration, June 2003, Section 5, SECTION 5 – Progressive Collapse Guidelines for Steel Frame Buildings, Table 5.1, page 5-17.

[x] "Design of Buildings to Resist Progressive Collapse," Unified Facilities Criteria UFC 4-023-03, Department of Defense, approved for public release, distribution unlimited, July 2004, Chapter 5, Structural Steel Design Requirements, Table 5-3, page 5-9.

SECTION 4
MITIGATION OF PROGRESSIVE COLLAPSE IN STEEL STRUCTURES

4.1. How is progressive collapse defined?

Progressive collapse is defined in the commentary of the American Society of Civil Engineers Standard ASCE 7-02 Minimum Design Loads for Buildings and Other Structures as “the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it.”[xi] The standard further states that buildings should be designed “to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage.” As discussed in the commentary of ASCE 7-02, “except for specially designed protective systems, it is usually impractical for a structure to be designed to resist general collapse caused by severe abnormal loads acting directly on a large portion of it. However, structures can be designed to limit the effects of local collapse and to prevent or minimize progressive collapse.”[xii] The structural design requirements presented herein were developed to ensure prudent precautions are taken where the event causing the initial local damage is undefined and the extent of initial damage is unknown.

Progressive collapse is a relatively rare event in the United States and other Western nations, as it requires both an abnormal loading to initiate the local damage and a structure that lacks adequate continuity, ductility, and redundancy to resist the spread of damage. These abnormal loads can occur as errors or problems during construction, as accidental impacts or energy releases anytime during the structure's life, or as intentional attacks by terrorists or other aggressors. A selection of incidents and photos of those incidents is shown in Figure 4.1. While the exact nature of the collapse (disproportionate or expected) is arguable in several of these

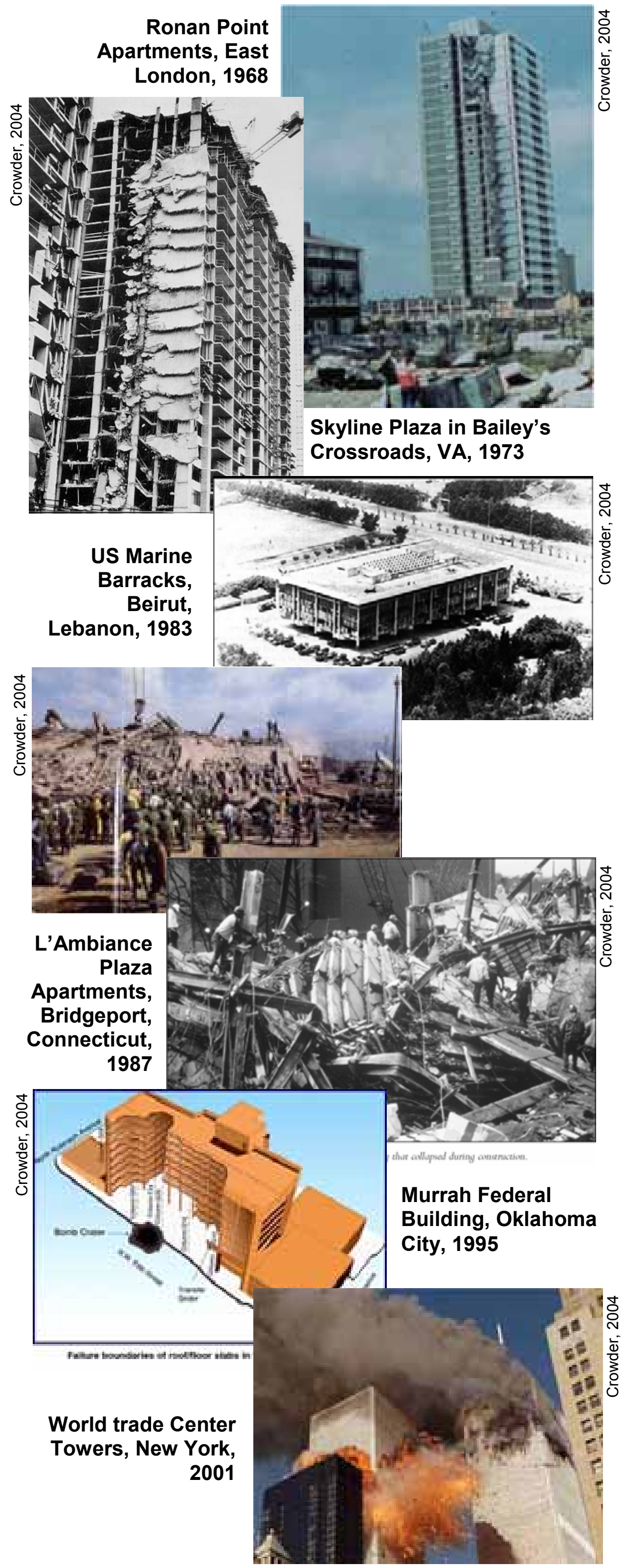


Figure 4.1
Selected Progressive Collapse Events; 1968-2001

cases, they all illustrate aspects of the mechanics we wish to further understand.

However the loads are generated, significant casualties can result when collapse occurs. This is illustrated by the April 19, 1995 bombing of the Alfred P. Murrah Federal Building in Oklahoma City, in which the majority of the 168 fatalities were due to the partial collapse of the structure and not to direct blast effects. The recent escalation of the domestic and international terrorist threat has increased the probability that other U.S. Government structures will be attacked with explosives or other violent means.

4.2. What are we really asking the structure to do under these extreme conditions?

We are not attempting to harden the structure to prevent failure of any particular component should it be loaded by blast or impact. We are seeking to limit the structural response to the local area severely damaged or failed by the direct attack. While specific threat information may allow designers to harden individual structural components against blast or impact, design to resist progressive collapse is "threat independent" in the sense that the designer "builds in" redundancy to survive these localized failures. Severe damage or component failure in this local damage area should not propagate, and produce overall structural response more severe than that produced by the initial attack. Thus, the desired performance of the structure involves activation of ductile and robust reserve capacity in the event of localized damage.

The most viable and practically proven alternative to control and limit the propagation of localized damage is the maintenance of structural continuity throughout the structure. Thus, as local damage occurs, surrounding structural components volunteer reserve capacity to span or “bridge” over the damaged area.

Alternatively, others have suggested that the concept of structural compartmentalization and isolation can be used to confine the damage to the structure to a small area. While this concept may be feasible, compartmentalization generally requires small spans and spaces with independent structural systems. This is significantly less efficient in terms of constructability and space usage, and is generally limited to special applications such as some military hardened structures. Hence, the continuity approach is advocated by current guidelines, and is presented here in two forms, tying and bridging. It is currently thought to be the more efficient and cost effective alternative solution to resisting progressive collapse.

4.3. Isn’t a concrete frame structural system better than a steel system?

The question posed in Section 3.0, “Are “efficient and readily erectable” structures and “blast-resistant” structures mutually exclusive terms?” addressed the perception that concrete construction performs better under blast loads than structural steel systems. When mitigating progressive collapse, the argument against structural steel is also unfair. As described in Section 4.2, we are asking the structure to prevent the propagation of local damage. This can be accomplished as readily in structural steel systems as in concrete systems when proper member selection and detailing is used. From the same reference cited in Section 3.0:

> “For steel structures, the frame has considerable ductility if the connections are properly designed and constructed.”[xiii]

Effective design to resist progressive collapse is again a matter of ductility and redundancy.

4.4. What is the difference between indirect and direct design for redundancy?

With Indirect Design, resistance to progressive collapse is considered implicitly "through the provision of minimum levels of strength, continuity and ductility".[xiv] The commentary in ASCE 7-02 goes on to present general design guidelines and suggestions for improving structural integrity. These include: 1) good plan layout, 2) integrated system of ties, 3) returns on walls, 4) changing span directions of floor slabs, 5) load-bearing interior partitions, 6) catenary action of the floor slab, 7) beam action of the walls, 8) redundant structural systems, 9) ductile detailing, 10) additional reinforcement for blast and load reversal, if the designer must consider explosive loads, and 11) compartmentalized construction.[xv] However, no quantitative requirements for either direct or indirect design to resist progressive collapse are provided in ASCE 7-02.

Direct Design approaches are those that include "explicit consideration of resistance to progressive collapse during the design process…"[xvi] These include: 1) the Alternate Path (AP) method, which requires that the structure be capable of bridging over a missing structural element, with the resulting extent of damage being localized, and 2) the Specific Local Resistance (SLR) method, which requires that the building, or parts of the building, provide sufficient strength to resist a specific load or threat.

While the alternate path or "bridging" method is referred to as a "direct design" approach, it should be recognized that the analysis is still, in a sense, threat independent; i.e., a single column or other vertical load carrying element is assumed to be "immaculately" removed. While it is true that this does not represent the actual blast or impact scenario where damage would be distributed and where more than one column could be removed, it is a method for introducing further redundancy and ductility into the design to resist progression of collapse.

4.5. What is "tying" and what are the tie forces to be determined?

Figure 4.2 illustrates how a system of mechanical ties could be used to provide continuity and redundancy in a structural frame.

In the tie force approach, the building is mechanically tied together, enhancing continuity, ductility, and development of alternate load paths. Tie forces are typically provided by the existing structural elements and connections that are designed using conventional design procedures to carry the standard loads imposed upon the structure.

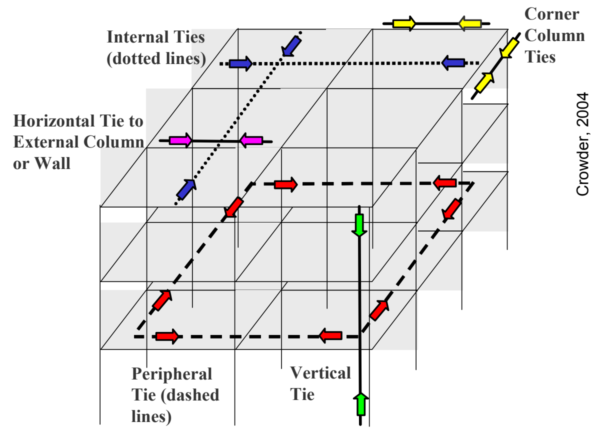


Figure 4.2
Schematic of Ties Required in Indirect Design to Resist Progressive Collapse

Ties can be provided as structural retrofits, but may be costly and aesthetically troublesome. Note that these "ties" are not synonymous with "ties" that enclose horizontal reinforcement in concrete structures (as defined in the 2002 version of the Building Code Requirements for Structural Concrete from the American Concrete Institute (ACI 318-02) for reinforced concrete design.)

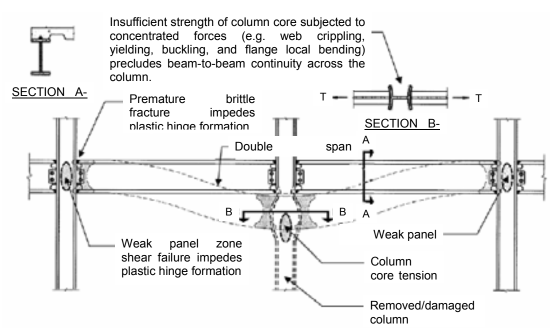
Depending upon the construction type, there are several horizontal ties that must be provided: internal, peripheral, and ties to edge columns, corner columns, and walls. Vertical ties are required in columns and load-bearing walls. Note that these "ties" are not synonymous with "ties" as defined in the 2002 version of the Building Code Requirements for Structural Concrete from the American Concrete Institute (ACI 318-02) for reinforced concrete design.

The load path for ***peripheral ties*** must be continuous around the plan geometry and, for ***internal ties***, the path must be continuous from one edge to the other. Along a particular load path, different structural elements may be used to provide the required tie strength, providing that they are adequately connected; for instance, internal tie strength may be provided by a series of beams on a beam line, provided that the connections to the intermediate elements (girders, beams or columns) can provide the required tie strength. Likewise, ***vertical ties*** must be continuous from the lowest level to the highest level. ***Horizontal ties to edge columns and walls*** do not have to be continuous, but they must be satisfactorily anchored back into the structure. For buildings that are composed of separate sub-structures or that incorporate expansion joints that create structurally independent sections, the tie force requirements are applied to each sub-structure or independent section, which are treated as separate units.

The only guidance currently prescriptively requiring ties in steel construction is the UFC 4-023-03, Unified Facilities Criteria (UFC), "Design of Buildings to Resist Progressive Collapse," which imposes horizontal and vertical tie force requirements for both low and high level of protection facilities for the DoD. Section 6.0 provides more detail on the calculation of tie forces.

4.6. How do you define "alternate paths" and what is "bridging"?

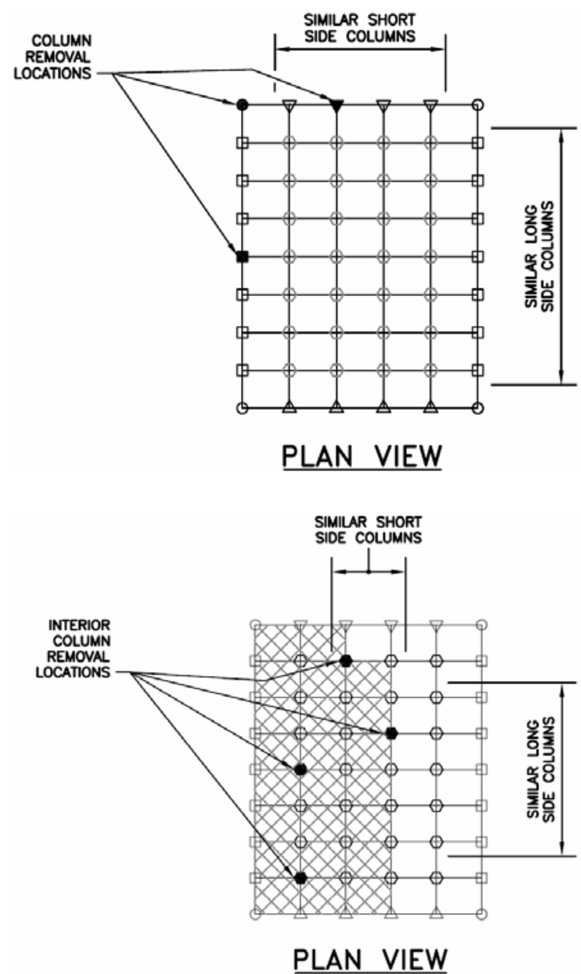
The direct design approach most widely used to evaluate a structure's potential to resist progressive collapse is termed the alternate path (AP) method. In this analysis approach, key structural members (typically a single column) are removed, and the structure analyzed to determine its capacity to span across or "bridge" across that "missing" member. Figure 4.3 illustrates the mechanism postulated when a damaged steel beam-column connection "bridges" over a lost column.



GSA Progressive Collapse Analysis and Design Guidelines, 2003

Figure 4.3
Illustration of Bridging Over a "Lost" Column Including Critical Transfer Mechanisms; Alternate Path Analysis Approach

Generally, and as a minimum, external columns must be removed near the middle of the short side, near the middle of the long side, and at the corner of the building. Columns must also be removed at locations where the plan geometry of the structure changes significantly, such as abrupt decrease in bay sizes and re-entrant corners, or, at locations where adjacent columns are lightly loaded, the bays have different tributary sizes, members frame in at different orientations or elevations, and other similar situations. Figure 4.4 presents the column removal strategy required by the DoD UFC.



UFC 4-023-03, 2004

Figure 4.4
Column removal Strategy per DoD UFC Progressive Collapse Criteria

AP analyses may be performed for just the first floor (GSA approach) or for each floor, one at a time (DoD approach). For the multi-floor approach, for example, if a corner column is specified as the removed element location, one AP analysis is performed for removal of the ground floor corner column; another AP analysis is performed for the removal of the first floor corner column; another AP analysis is performed for the second floor corner column, and so on. If the designer can show that similar structural response is expected for column removal on multiple floors (say, floors 4 though 10), the analysis for these floors can be omitted but the designer must document the justification for not performing these analyses.

For structures with underground parking or other uncontrolled public ground floor areas, it is also recommended that internal columns be removed near the middle of the short side, near the middle of the long side and at the corner of the uncontrolled space. The removed column extends from the floor of the underground parking area or uncontrolled public ground floor area to the next floor (i.e., a one-story height must be removed). Internal columns must also be removed at other critical locations within the uncontrolled public access area, as determined with engineering judgment.

For both external and internal column removal, continuity must be retained across the horizontal elements that connect to the ends of the column.

4.7. What connection attributes are critical to good performance?

The following connection attributes should provide the increased ductility and robustness needed to resist blast effects and reduce the potential for progressive collapse, when used in a well-designed global structural system.

- Clearly defined beam-to-beam continuity
- Connection redundancy
- Connection resilience
- Reliable inelastic connection rotational capacity
- Concurrent reserve axial tension capacity

4.8. What is beam-to-beam continuity?

Beam-to-beam continuity consists of a distinct, clearly defined link across a column. This link must be capable of independently transferring gravity loads for a removed column condition, regardless of the actual or potential damage state of the column.

4.9. What is connection resilience and why is it required?

A beam-to-column connection should provide direct, multiple load paths through the connection and should exhibit the ability to withstand the rigors of destructive loading conditions that accompany severe blast damage of connecting members, including column damage or removal, without rupture. This ability is facilitated by the connection's torsional and weak-axis flexural strength, its robustness, and its primary use of proven ductile properties of a given construction material.

4.10. How is inelastic rotational capacity provided in a connection design?

Indirect and direct design methods work together when structural members and connections progress through two overlapping mechanisms as they attempt to "bridge" over a destroyed component. First, existing moment capacity and inelastic rotational resistance is contributed in flexure as the component and connection deforms (bridging). Second, the axial capacity of the member and connection are "activated" as the system attempts to reach equilibrium through catenary action.

Until more definitive tests have been conducted on the performance of steel connections, including the effects of moment and axial tension interaction, steel connections should meet ANSI/AISC 341-02 standards for inelastic rotational capacity. This should ensure minimum performance characteristics for steel moment connections.

In extreme loading conditions connections may be required to withstand significant inelastic rotations. Because of their proven ductility, only steel frame beam-to-column connection types that have been qualified by full-scale testing to verify that they provide the required level of connection rotational capacity should be used in the design of new buildings to mitigate blast effects and progressive collapse. Connections that have satisfied the SMF rotational capacity provisions of AISC 341-02 should provide the necessary rotational capacity.

4.11. Why is reserve axial tension capacity required in connections participating in designs to resist progressive collapse?

If the blast or impact loading is large enough to compromise the vertical load carrying capacity of the column, transfer girder, or other gravity load carrying element, post blast development of large tension loads in the beams due to large displacements generated by a double-span condition is likely to occur. In such cases the beams start acting less like beams (i.e., resisting vertical loads through bending) and more like cables, resisting vertical loads through catenary action, resulting in post-yield interaction of moment and axial tension in both the beams and their beam-to-column-to-beam connections.

In order to reach a state in which catenary action is achieved, connections must first go through significant inelastic rotations while resisting increasing levels of axial tension. The axial tension portion of the applied load will increase as the frame response goes from resisting flexure to resisting a combination of moment and axial tension, and ultimately to resisting primarily tensile loads.

[xi] "Minimum Design Loads for Buildings and Other Structures," SEI/ASCE 7-02, 2003, Section C1.4, General Structural Integrity, page 232.
[xii] ASCE 7-02, page 232.
[xiii] "Lessons From the Oklahoma City Bombing – Defensive Design Techniques," Chapter 19, "Structural Countermeasures", pages 33-34, ASCE, 1997.
[xiv] ASCE 7-02, page 234.
[xv] ASCE 7-02, page 234.
[xvi] ASCE 7-02, page 234.

SECTION 5
BEST PRACTICES TO MITIGATE BLAST EFFECTS

5.1. Do any of the design standards (AISC, AISI, ACI) or building codes (IBC, UBC) specifically address blast effects?

For the most part, no. There are some blast related guides such as ASCE's "Design of Blast Resistant Buildings in Petrochemical Facilities", but, generally, blast design has historically been a specialized field where structural and mechanical engineers experienced with military manuals and software perform the necessary load calculations and dynamic analyses.

Several current efforts are being pursued including AISC's "Steel Design Guide: Mitigation of Blast and Progressive Collapse,"[xiii] and ASCE's "Standard for Blast Protection of Buildings".

5.2. What manuals and guidelines do exist? Are they publicly available?

There are two classes of documentation available: educational references and documents containing engineering design guidance.

Some recent and excellent educational references are publicly available. Two of the best are FEMA's (the Federal Emergency Management Agency) FEMA 427, "Risk Management Series Primer for Design of Commercial Buildings to Mitigate Terrorist Attacks" and FEMA 426, "Risk Management Series Reference Manual to Mitigate Potential Terrorist Attacks Against Buildings". Several very good educational websites also exist. One of the best is provided by the National Institute of Building Sciences (NIBS) at www.wbdg.org. (Figure 5.1).

www.wbdg.org

Figure 5.1
NIBS Whole Building Design Guide Security Pages

The three U.S. engineering manuals that contain blast-resistant design information for steel structures are primarily produced by the DoD. A short description of each and its availability is listed below:

- ***TM 5-1300, "Structures to Resist the Effects of Accidental Explosions," Departments of the Army, Navy and Air Force, 1990****, approved for public release*—the seminal publicly available design guide for blast-resistant design. Somewhat dated now (over 14 years old) since significant research into conventional weapons effects research has been conducted in the last decade. Probably the most comprehensive set of design information for steel structures subjected to blast loads.

- ***UFC 3-340-01, "Design and Analysis of Hardened Structures to Conventional Weapons Effects", 2002****, for official use only, distribution limited to authorized U.S. Government agencies and their contractors, export controlled*—the most recent and comprehensive guide for blast-resistant design produced by the DoD. Unfortunately, contains little new, and, in fact, less material on steel design than does TM 5-1300.

- ***TM 5-853-1/2/3, "Security Engineering Project Development, Security Engineering Concept Design, and Security Engineering Final Design," 1994**, for official use only, distribution limited to authorized U.S. Government agencies and their contractors*—a very useful document, soon to be published in updated form as UFC 4-011-02.

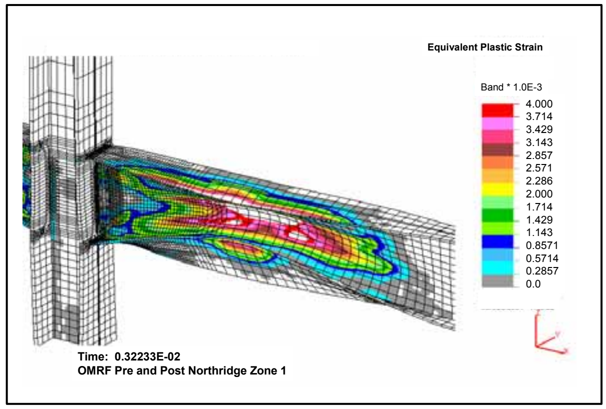
Many other manuals and guidelines exist that are derivative of these two documents. Many provide additional detail on blast load prediction and weapons effects, but none provide more detail related specifically to steel structure design.

5.3. What are the performance criteria for blast-loaded steel structures?

Performance criteria for steel structure response to blast loads are somewhat component dependant (i.e., beams, columns, connections, plates, etc. all have distinct criteria), but they are generally expressed as "response limits" in terms of member rotations based on dynamic flexural response modes. In other words, the best existing response criteria applicable to dynamic analysis and design are based on bending in sections. Clearly, other response modes and potential failure modes occur and must be considered, but present criteria only address those modes through static capacity calculations or empirically derived rotation criteria. Figure 5.2 illustrates the complex interaction of response modes occurring in a steel beam directly loaded with explosive blast. Standard response limits ignore much of this detail.

Figure 5.3 presents response limit information from UFC 3-340-01.

Shear criteria for blast loads, for example, generally are based on existing static shear capacity calculations for sections per AISC requirements. A similar approach is taken for local instability (web crippling) and member stability (bracing requirements). The general approach in design is to ensure that the flexural resistance to load provided by the member is not limited by these failure modes with limited ductility. If shear and stability considerations are satisfied, then the member is assumed to have sufficient capacity to undergo the plastic rotations permitted by the response criteria.



Karns and Houghton, 2004

Figure 5.2
Computational Example of Steel Member Vulnerability to Local Blast; Blast Induced Buckling Criteria from UFC 3-340-01

Structural Element	Response Limit		
	Support Rotation, θ rad (deg)	Ductility, μ	Other
Reinforced Concrete			
Slabs and Beams			
Unrestrained	0.105 (6)	-	
Restrained-Light Damage	0.105 (6)	-	
Restrained-Moderate Damage	0.21 (12)	-	
Restrained-Heavy Damage	0.349 (20)	-	
Deep Slabs and Beams			
Unrestrained	0.035 (2)	-	
Restrained-Light Damage	0.105 (6)	-	
Restrained-Moderate Damage	0.21 (12)	-	
Structural Steel			
Beams	0.21 (12)	20	
Plates	0.21 (12)	40	
Frames	0.035 (2)	-	H/25 Sidesway
Blast Doors (Reinforced Concrete or Structural Steel)			
Operable Postevent	0.017 (1)	3	
Non-Operable Postevent	Use reinforced concrete and structural steel limits above		

UFC 3-340-01, 2002

Figure 5.3
Table of Allowable Member and Connection Rotation

Dynamic shear resistance of wide flange shapes has traditionally been thought sufficient (shear failure difficult to achieve) except for baseplate connections, where shear failure must be prevented through embedded plates (concrete floor or slab providing shear resistance) or through enhanced baseplate details. The use of larger and heavier structural shapes in high rise and high moment demand structural systems may invalidate this assumption, however.

Performance criteria for connections is currently almost exclusively based on empirical data from experiments conducted to support seismic designs. Results from the SAC (SEAOC/ATC/CUREe) connection research project conducted after the Northridge earthquake provide the most current information related to plastic connection performance. Lessons learned from these tests are essential for ductile detailing for blast-resistant design. The SAC tests and other post-Northridge research programs evaluated several specific steel beam-to-column connections considered appropriate for seismic design. The research resulted in approaches for capacity calculation and rotation performance for these connections. The reader should be cautioned that all of these tests were conducted using cyclic static loads without significant axial load applied to the connection. The cyclic nature of the load may lend some conservatism to the interpretation of the results, but clearly the absence of axial loads does not.

Figure 5.4 shows the response of a "coverplated" connection from tests performed as a part of the SAC series. The recommended equation for allowable rotation is also presented.

TM 5-1300 contains an extensive section on steel design to resist blast loads. While dated, this information includes design guidance for plates, cold-formed sections and even steel blast doors, in addition to information on beam and column design.

5.4. What material properties of steel are important to consider in blast-resistant design?

Ductility and rate sensitivity are the two key parameters when considering the performance of structural steel in blast design. Structural steel generally exhibits a linear tensile stress-strain relationship up to the proportional limit, which is either close to or identical to the yield point.

Karns, 2004 and FEMA 355D, 2000

Figure 5.4
SAC Tests and Rotation Capacity Equation for SidePlate™ Connection

$\Theta_p = 0.056 - 0.0011 d_b$

Beyond the yield point, structural steel can stretch substantially through plastic yielding without an appreciable increase in stress, with the amount of elongation reaching 10 to 15 times that needed to reach yield, a range that is termed the yield plateau. Beyond that range, strain hardening occurs, i.e., additional elongation is associated with an increase in stress. After reaching a maximum nominal stress called the tensile strength, a drop in stress occurs at an elongation (at rupture) amounting to 20 to 30 percent of the specimen's original length. Therefore, structural steel has substantial ductility for use in blast-resistant design. Figure 5.5 presents the stress-strain curves for a number of steels.

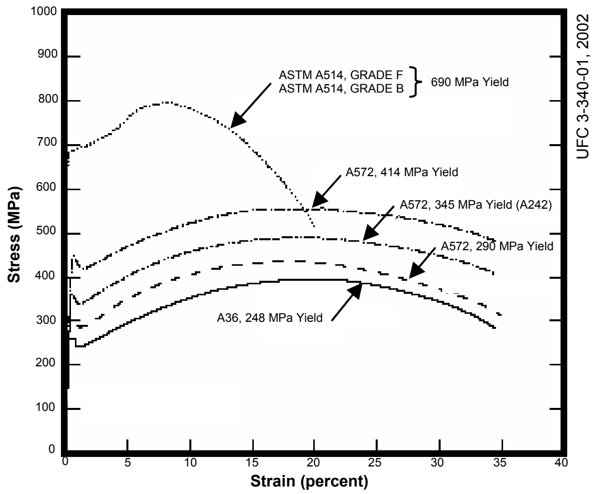


Figure 5.5
Tensile Stress-Strain Curves for Typical Steels

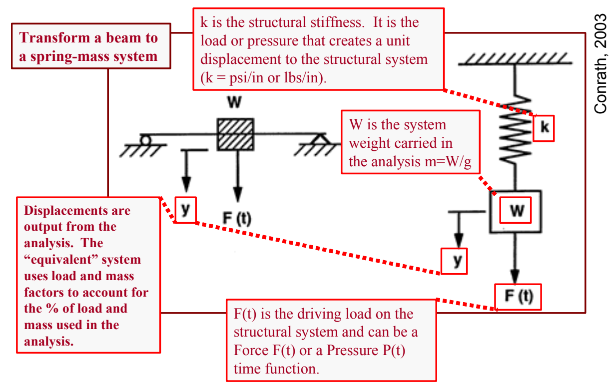


Figure 5.6
Single-Degree-of-Freedom (SDOF) Formulation

Structural steel yield strengths and to a lesser degree ultimate strengths are also dependant on strain rate, or the speed at which the steel deforms axially, in bending or in shear. Figure 5.8 illustrates strain rate sensitivity for several structural steels.

While beyond the scope of this document, particular attention must be paid to weld metal placement in connection design for steel members.

5.5. What are the accepted dynamic analysis approaches?

Traditionally, blast-resistant design of individual structural components or simple assemblages of components has been accomplished using single-degree-of-freedom (SDOF) analysis.

SDOF analysis is a non-linear dynamic analysis that is simplified through the definition of a single response mode for a dynamic system. In other words, an SDOF analysis assumes a response mode and a response shape (flexural response with a 3 hinge mechanism for a fixed beam, for example). Figure 5.6 presents a schematic and notes describing the basic components of an SDOF system.

The simple spring-mass system shown in Figure 5.6 can also include damping, although peak displacements are generally not effected. SDOF analyses are fairly straightforward but their simplicity can cause a designer to overlook potential failure and response modes not initially assumed in the SDOF derivation.

In recent years, other sophisticated analysis tools have matured and have become more accessible (running on personal computers). Computer Software such as ANSYS, ADINA, ABAQUS, NISA II and LSDYNA, to name just a few, allow full finite element formulations of structural components and structural systems to be developed and evaluated. This "micro" level analysis allows individual structural members and connections to be considered in detail, and performance characteristics developed (stiffness, ductility, failure modes) either as a part of a larger finite element formulation, or as input to a "macro" model of a structural system. These "macro" models can be developed with software tools such as SAP/ETABS, STAAD PRO/LARSA, and RAM PERFORM where member characteristics (nonlinear properties) developed through SDOF or FEA "micro" models are provided as inputs and where structural system analysis and design can be performed.

Simple approaches, like the pressure-impulse (*P-i*) curves mentioned previously, which capture component nonlinearity and dynamic response but have further restrictions (fixed load shape and response limits), are also available. Recent research[xiv] has shown these simple tools to be accurate in some scenarios, but possibly even unconservative in others. Figure 5.7 illustrates some comparisons between a *P-i* representation of a steel column and some LSDYNA analysis showing that representation to be unconservative.

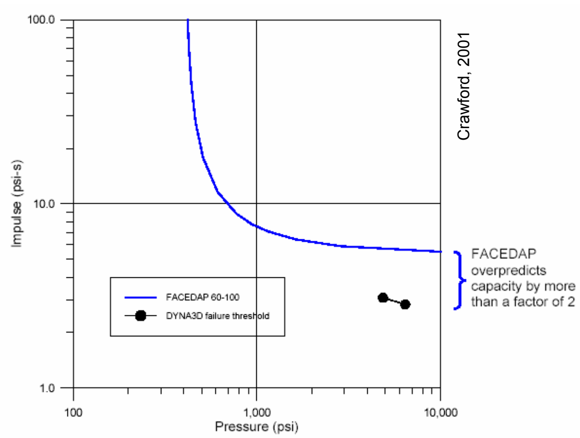


Figure 5.7
Detailed Numerical Analysis (DYNA-3D) Illustrates Unconservatism in Some Simple Approaches; W12x87 Steel Column Comparison

5.6. What are the specific steps involved in an analysis for blast loads?

Because blast analysis and design of structural components involves a full dynamic and material (and possibly geometric) nonlinear analysis, even a "simplified" analysis approach, such as an SDOF analysis, is fairly complex. The example below illustrates many of the steps required in the evaluation of a "simple" member. Many of these steps have been incorporated into the automated routines available in some of the software tools (SPan32, FEMA's NONLIN and the Corps of Engineer's SBEDS programs), but are included here for illustration.

A simplified analysis of a typical structural component such as a gravity load bearing floor beam in a steel beam-composite slab system (responding in flexure) involves the following steps:

- *Determine beam loads by calculating blast loads applied directly to the floor system (CONWEP), or applied indirectly through propagation through exterior building openings (multiple reflections model such as BlastX or CFD approach) and then:*

  1) *accumulating contributory floor slab loads, or*
  2) *applying the full resistance time history of the floor slab system to the beam, or*
  3) *analyzing the floor slab system separately and determining dynamic reactions*

  (Determining the dynamic reactions of the slab system may result in a more efficient design, as reactions are the sum (a time history) of a portion of the slab resistance during response and the slab applied loads)

- *Determine beam properties and perform "static" checks:*

  1) *determine dynamic material strengths including "overstrength" (multiplier to account for actual strength) and dynamic increase factor (to account for strain rate effects)* (see Figure 5.8 for examples of overstrength and rate factors to be used in analysis)
  2) *determine the mass of the responding system (including contributory slab system if dynamic reactions or time history of resistance from slab system is* <u>*not*</u> *used)*
  3) *calculate moment capacity of the beam—assumed to be fully plastic section for response ductilities >3*
  4) *check flange and web thickness requirements for local buckling*
  5) *check lateral bracing requirements*
  6) *check shear capacity of the section—generally considered to be provided by the wide flange web, but larger/heavier sections may require a check the effect of shear on available moment capacity*

7) *check axial load and bending interaction if significant tensile loads are present*

Property	Year	Specification	Factor
Tensile Strength	Prior to 1961		1.10
Yield Strength	Prior to 1961		1.10
Tensile Strength	1961-1990	ASTM A36/A36M-001	1.10
	1961-present	ASTM A572/A572M-89, Grp 1	1.10
		ASTM A572/A572M-89, Grp 2	1.10
		ASTM A572/A572M-89, Grp 3	1.05
		ASTM A572/A572M-89, Grp 4	1.05
		ASTM A572/A572M-89, Grp 5	1.05
	1990-present	ASTM A36/A36M-001 & Dual Grade Grp 1	1.05
		ASTM A36/A36M-001 & Dual Grade Grp 2	1.05
		ASTM A36/A36M-001 & Dual Grade Grp 3	1.05
		ASTM A36/A36M-001 & Dual Grade Grp 4	1.05
Yield Strength	1961-1990	ASTM A36/A36M-001	1.10
	1961-present	ASTM A572/A572M-89, Grp 1	1.10
		ASTM A572/A572M-89, Grp 2	1.10
		ASTM A572/A572M-89, Grp 3	1.05
		ASTM A572/A572M-89, Grp 4	1.10
		ASTM A572/A572M-89, Grp 5	1.05
	1990-present	ASTM A36/A36M-001 & Dual Grade Grp 1	1.10
		ASTM A36/A36M-001 & Dual Grade Grp 2	1.05
		ASTM A36/A36M-001 & Dual Grade Grp 3	1.10
		ASTM A36/A36M-001 & Dual Grade Grp 4	1.05
		ASTM A36/A36M-001 & Dual Grade Grp 5	1.05
Tensile Strength	All	Not listed[1]	1.10
Yield Strength	All	Not listed[1]	1.10

1. For materials not conforming to one of the listed specifications

GSA Progressive Collapse Analysis and Design Guidelines, 2003

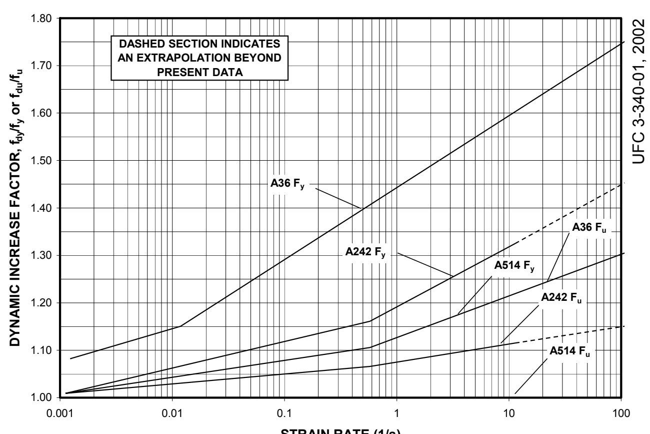


Figure 5.8
Material Overstrength Factors and Strain Rate Factors for Structural Steel from UFC 3-340-01

- *Determine flexural resistance function and related properties for the section:*

1) *determine elastic (and elasto-plastic) stiffnesses and deformation at maximum resistance ("yield point")*
2) *determine non-linear resistance based on restraint conditions (end conditions) of the beam—may include hardening and tensile membrane capacity enhancement* (see Figure 5.9 for examples of stiffness and resistance calculations from UFC 3-340-01)

3) *calculate load and mass factors for SDOF analysis—the response shape assumed in SDOF analysis based on end conditions determines the multipliers that are applied to the mass, load, stiffness and resistance of the system to create the "equivalent" SDOF system* (see Figure 5.10 for typical load-mass factors, again from UFC 3-340-01)

- *Perform the SDOF analysis and check maximum rotations/ductilities against member response limits:*

1) *SDOF midspan deflection/rotation should not exceed protection level response limit*
2) *Rotations at the support should not exceed those specified for the connection*

(Response limits are currently defined for structural steel and connections in UFC 3-340-01, however, a new and more comprehensive table of response limits is to be included in UFC 4-020-02, "Security Engineering Facility design Manual."

- *Check beam reactions(maximum resistance) against shear capacity of connection*

UFC 3-340-01, 2002

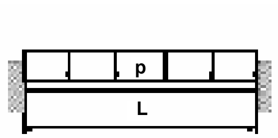
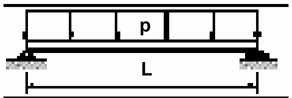
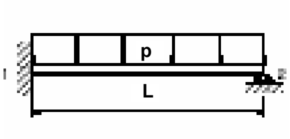
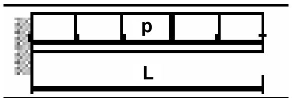
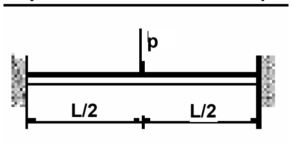
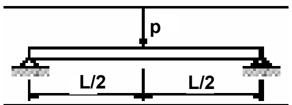
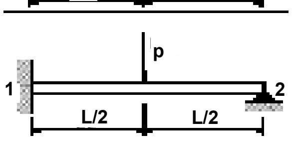
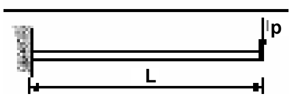
Support Conditions and Loading Diagrams	Elastic Resistance	Ultimate Flexural Resistance	Stiffness: Bending	Stiffness: Shear	Equivalent Support Shears
	$r_e = \frac{12M_n}{L^2}$	$r_u = \frac{8(M_n + M_p)}{L^2}$	$k_e = \frac{384EI}{L^4}$ $k_{ep} = \frac{384EI}{5L^4}$ $k_E = \frac{307EI}{L^4}$	$k_e = k_{ep} = \frac{hE}{0.35L^2}$	$V = \frac{r_u L}{2}$
	$r_e = r_u$	$r_u = \frac{8M_p}{L^2}$	$k_e = k_E = \frac{384EI}{5L^4}$	$k_e = \frac{hE}{0.35L^2}$	$V = \frac{r_u L}{2}$
	$r_e = \frac{8M_n}{L^2}$	$r_u = \frac{4(M_n + 2M_p)}{L^2}$	$k_e = \frac{185EI}{L^4}$ $k_{ep} = \frac{384EI}{5L^4}$ $k_E = \frac{160EI}{L^4}$	$k_e = k_{ep} = \frac{hE}{0.35L^2}$	$V_1 = \frac{5r_u L}{8}$ $V_2 = \frac{3r_u L}{8}$
	$r_e = r_u$	$r_u = \frac{2M_n}{L^2}$	$k_e = k_E = \frac{8EI}{L^4}$	$k_e = \frac{hE}{1.4L^2}$	$V = r_u L$
	$r_e = \frac{8M_n}{L}$	$r_u = \frac{4(M_n + M_p)}{L}$	$k_e = \frac{192EI}{L^3}$ $k_{ep} = \frac{48EI}{L^3}$ $k_E = \frac{192EI}{L^3}$	$k_e = k_{ep} = \frac{hE}{0.7L}$	$V = \frac{R_u}{2}$
	$r_e = r_u$	$r_u = \frac{4M_p}{L}$	$k_e = k_E = \frac{48EI}{L^3}$	$k_e = \frac{hE}{0.7L}$	$V = \frac{R_u}{2}$
	$r_e = \frac{16M_n}{3L}$	$r_u = \frac{2(M_n + 2M_p)}{L}$	$k_e = \frac{107EI}{L^3}$ $k_{ep} = \frac{48EI}{L^3}$ $k_E = \frac{106EI}{L^3}$	$k_e = k_{ep} = \frac{hE}{0.7L}$	$V_1 = \frac{11r_u}{16}$ $V_2 = \frac{5r_u}{16}$
	$r_e = r_u$	$r_u = \frac{M_n}{L}$	$k_e = k_E = \frac{3EI}{L^3}$	$k_e = \frac{hE}{2.8L}$	$V = r_u$

Figure 5.9
Resistance and Stiffness Values for SDOF Analysis

UFC 3-340-01, 2002

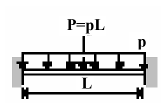
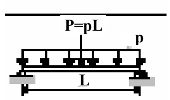
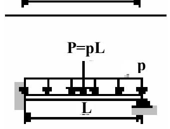
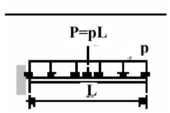
Loading Diagram	Strain Range	Load Factor K_L	Mass Factor K_M	Load-Mass Factor K_M/K_L	Dynamic Reaction V
	Elastic	0.53	0.41	0.77	0.36R + 0.14P
	Elastoplastic	0.64	0.50	0.78	0.39R + 0.11P
	Plastic	0.50	0.33	0.66	$0.38R_m$ + 0.12P
	Elastic	0.64	0.50	0.78	0.39R + 0.11P
	Plastic	0.50	0.33	0.66	$0.38R_m$ + 0.12P
	Elastic	0.58	0.45	0.78	0.43R + 0.12P
	Elastoplastic	0.64	0.50	0.78	0.39R+0.11P- M_{ps}/L
	Plastic	0.50	0.33	0.66	$0.38R_m$+0.12P- M_{ps}/L
	Elastic	0.40	0.26	0.65	0.69R + 0.31P
	Plastic	0.50	0.33	0.66	$0.75R_m$ + 0.25P

Figure 5.10
Load and Mass Factors and Dynamic Reactions for SDOF Analysis

5.7. What are the best approaches for improving the performance (designing in "hardness") of members and connections?

Ductility is as important as stiffness or strength, and ductility is as important as mass or inertial resistance. Efficient sections for resistance of blast will balance connection capacity and ductility with structural section resistance and ductility.

The list below includes some other general guidelines for blast-resistant design of steel members and connections:

- Ensure that beam local buckling or shear failure will not occur prior to development of full plastic moment capacity
- Provide full length support of beams to prevent lateral torsional buckling using slab on metal deck floor slabs (use shear studs instead of puddle welds), or similar
- Use "seismically compact" beam sections if possible
- Attempt to ensure full plastic moment development in beams in both positive and negative directions (braced accordingly)
- Use beam-to-beam connection designs that incorporate the essential connection attributes described in Section 4.7, including the rotational capacity requirements specified in AISC 341-02
- Check column stability for greater unbraced length due to loss of floors providing lateral support
- Consider the use of concrete-filled HSS or pipe or concrete-encased wide flanges as columns—mitigates local buckling issues
- Use strong column-weak beam approach to ensure beam/connection hinging prior to column plastic hinge formation
- Embed column base plates in foundation slab or otherwise reinforce base plate connections for shear/reaction loads

5.8. What types of retrofit or "hardening" techniques are available for existing structures?

A variety of approaches can be used to harden existing structural steel systems against blast. Some general concepts are provided below:

- Protect key members—use architectural treatments to increase standoffs for "satchel" type explosive charges
- Encase wide flange columns in concrete per AISC seismic provisions
- Harden floor slab systems (supplemental composite slab with top steel, composite appliqué, etc.) to lessen chance of loss of lateral stability
- Supplement existing structural system—add beams between girders to shorten floor system spans
- Improve connection performance (ductility) using upgraded beam-to-beam connection designs that incorporate the essential connection attributes described in Section 4.7

[xvii] AISC anticipates work on this guide to begin in late 2004, with publication within 18 months.

[xviii] Crawford, J.E., et al, "Design Studies Related to the Vulnerability of Office Buildings to Progressive Collapse Due to Terrorist Attack," Karagozian and Case TR-01-10.1, October 2001.

SECTION 6
BEST PRACTICES TO MITIGATE PROGRESSIVE COLLAPSE EFFECTS

6.1. Do any of the design standards (AISC, AISI, ACI) or building codes (IBC, UBC) specifically address progressive collapse?

Other than the general requirements for structural integrity in ASCE 7-02 and in ACI 318-02 mentioned previously, there are no specific public domain or public law requirements (citing design guides by reference) for progressive collapse mitigation in the design of structures.

6.2. What manuals and guidelines do exist? Are they publicly available?

There are two primary U.S. Government guidelines that address provisions for the mitigation of progressive collapse in structural designs. These are the 2003 "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects" published by the U.S. General Services Administration and the 2004 (final draft) of UFC 4-023-03, "Design of Buildings to Resist Progressive Collapse", published by the Department of Defense (Figure 6.1). Both of these documents are approved for public release without restrictions.

Many other (primarily Government) guides and documents reference the GSA and UFC criteria, including UFC 4-010-01, "DoD Minimum Antiterrorism Standards for Buildings", October 2003, the "ISC Security Design Criteria for New Federal Office Buildings and Major Modernization Projects", May 2001 (applicable to the construction and modernization of general purpose office buildings and court facilities occupied by Federal employees in the United States not under the jurisdiction of the DoD), the "Federal Reserve System Facility and Security Guidelines", June 2002, the General Services Administration's "Facilities Standards for the Public Buildings Service", and others.

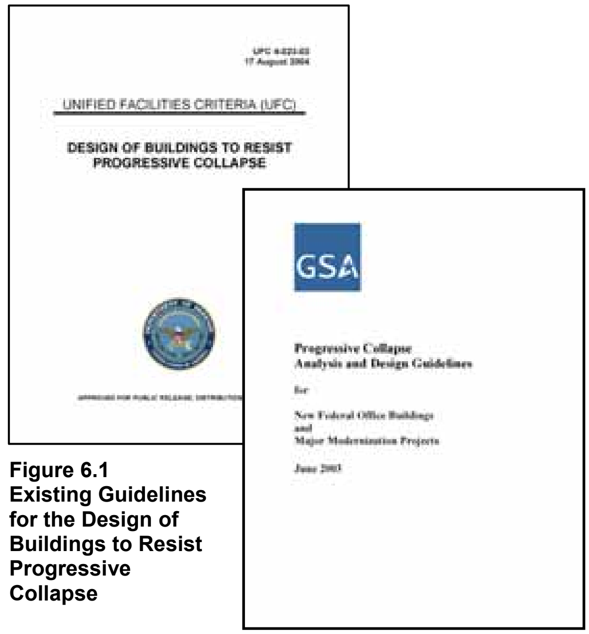


Figure 6.1
Existing Guidelines for the Design of Buildings to Resist Progressive Collapse

6.3. What are the performance criteria for progressive collapse mitigation in steel structures?

As previously introduced, and as based on the best research and experience (primarily in the U.K.) to date, two methods for the mitigation of progressive collapse are available in the UFC and GSA guidelines. The GSA approach recommends the ***direct design*** alternate path or "bridging" approach exclusively. The UFC prescribes the ***indirect design*** approach of using ties for lower levels of protection, and the alternate path approach should tying not be possible or for higher levels of protection.

The ***tying approach*** presumes that reserve axial capacity in members and connections is sufficient to:

- Allow structural members to span over "lost" vertical support through catenary action, assuming a deflection of 10 percent of the "doubled" span

- Provide distributed and reserve strength to “carry” the reactions from these “doubled” spans, and
- to support components through supplemental strength (such as continuous ties perpendicular to primary slab steel) whose primary design support condition has failed

The tying approach is a prescriptive methodology used to provide continuity in a structural system. The mechanics behind this approach are not well defined, but it has been applied in U.K. Building Codes for over 20 years. The primary assumption underlying the effectiveness behind this method is that structural members and their connections have sufficient rotational ductility to allow axial capacity to be developed in the form of a catenary at fairly large "double span" deflections. Deflections of as much as 10 percent of the double span length are assumed when "minimum" catenary forces are calculated. There is some concern that steel frame connections have not reliably achieved such large rotations in seismic connection testing (pre- and post-Northridge).

Ties consist of internal, wall/column to internal, peripheral and vertical continuous connection. As previously described, ties are theorized to work by providing a tensile capacity available in members and connections after flexural response (bending and load redistribution) has occurred in a structural system. Figure 6.2 provides a schematic of tie action in a structural steel frame.

The UFC requires that:

“All buildings must be effectively tied together at each principal floor level. Each column must be effectively held in position by means of horizontal ties in two directions, approximately at right angles, at each principal floor level supported by that column. Horizontal ties must similarly be provided at the roof level, except where the steelwork only supports cladding that weighs not more than 0.7 kN/m^2 (14.6 lb/ft^2) and carries only imposed roof loads and wind loads.

"For steel buildings, continuous lines of ties must be arranged as close as practical to the edges of the floor or roof and to each column line…. At re-entrant corners, the tie members nearest to the edge must be anchored into the steel framework….”[xix]

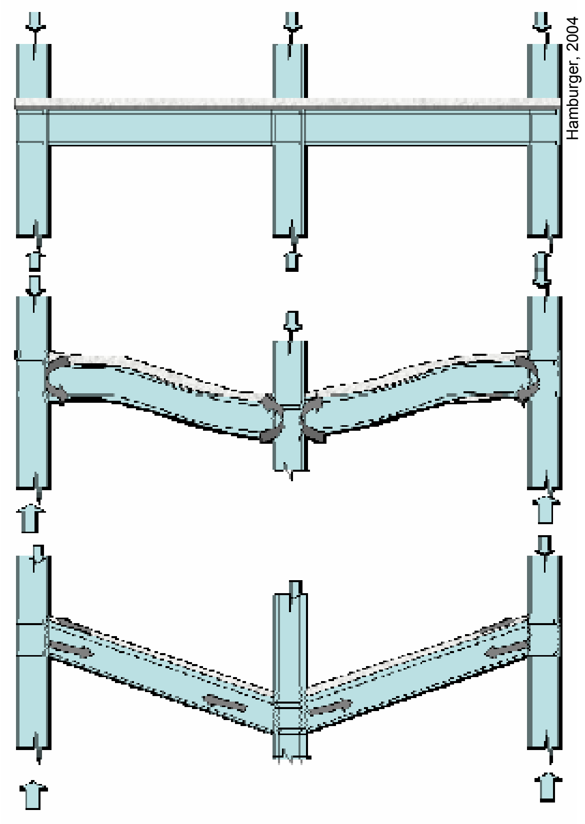


Figure 6.2
Steel Frame Progression from Flexural Response to Tensile Catenary Action

Horizontal ties are continuous ties extending across the structure in two perpendicular directions at each floor level, and can consist of steel members that are a part of the structural frame. Steel members acting as internal ties and their connections are to be capable of resisting the following required tie strength, which need not be considered as additive to other loads:

- $(1.2D + 1.6L)\, s_t\, L_l$;
  but not less than 16.9 kips

where:
D = Dead Load (lb/ft^2)

L = Live Load (lb/ft^2)
L_l = Span (ft)
s_t = Mean transverse spacing of the ties adjacent to the ties being checked (ft)

Peripheral ties are continuous around the perimeter of the structure at each floor level. Peripheral ties must be capable of resisting:

- $0.25\ (1.2D + 1.6L)\ s_t\ L_l$;
  but not less than 8.4 kips

Horizontal ties to perimeter columns or walls anchoring the column nearest to the edges of a floor or roof and acting perpendicular to the edge have required tie strengths equal to:

- the greater of the load specified for internal ties or

- 1percent of the maximum factored vertical dead and live load in the column that is being tied, considering all load combinations used in the design.

Vertical ties are prescribed so that all columns are continuous through each beam-to-column connection. All column splices must provide a design tie strength equal to the largest factored vertical dead and live load reaction (from all load combinations used in the design) applied to the column at any single floor level located between that column splice and the next column splice down or the base of the column.

Figure 6.3 presents a plan view of the location of required ties.

Should tying not be practical or should higher levels of protection be required, the UFC requires that an ***alternate path analysis*** be conducted. Similarly, the GSA criteria require a slightly varied form of the alternate path approach for the first floor perimeter columns.

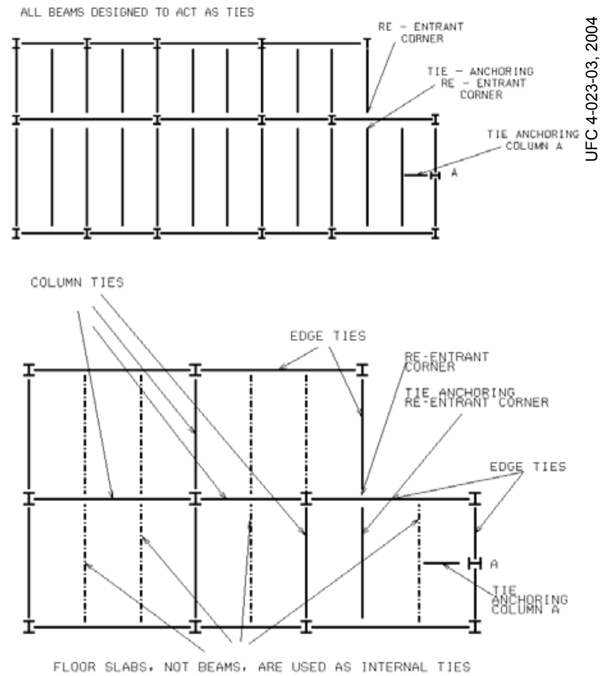


Figure 6.3
Plan Schematics of the Application of Steel Building Components as Ties for Progressive Collapse

The ***LRFD-based UFC criteria*** are expanded upon below first for further illustration.

In AP analysis, the ***acceptable extent of damage*** for the removal of a wall or column on the ***external envelope*** of a building greater than three stories in height, is:

- a collapsed area of the floor directly above the removed element less than the smaller of 70 m^2 (1500 ft^2) or 15 percent of the total area of that floor (the same requirement applies to the floor directly beneath the removed element) (GSA is 1800 ft^2)

- for three story buildings, the damage may extend to all three floors unless more stringent requirements are stipulated by the facility owner. In addition, any collapse must not extend beyond the structure tributary to the removed element.

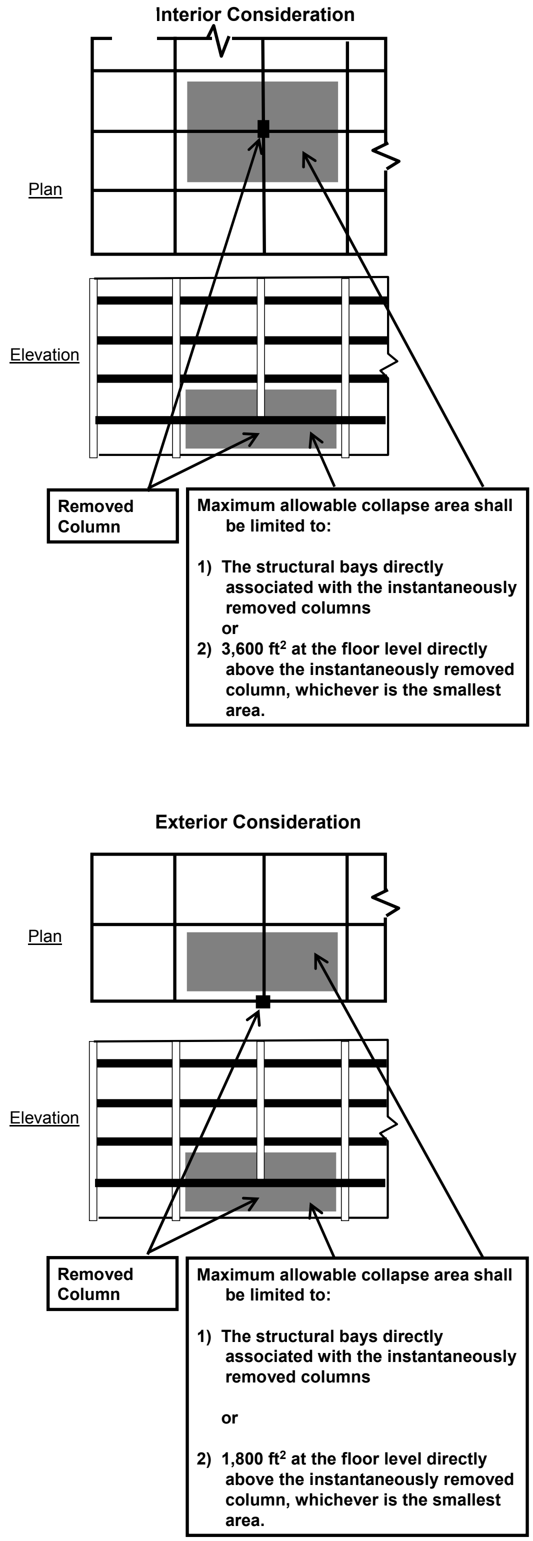


Figure 6.4
Acceptable Damage Extent for External and Internal Alternate Path Approach Column Removal Scenarios

For the removal of an ***internal wall or column*** of a building greater than three stories in height, the damage limits require that:

- the collapsed area of the floor directly above the removed element must be less than the smaller of 140 m^2 (3000 ft^2) or 30 percent of the total area of that floor (the same requirement applies to the floor directly beneath the removed element) (GSA is 3600 ft^2)

- for three story buildings, the damage may extend to all three floors unless more stringent requirements are stipulated by the facility owner. In addition any collapse must not extend beyond the bays immediately adjacent to the removed element.

Figure 6.4 illustrates the allowable extent of damage and collapse per the UFC.

The acceptability criteria for the AP method consist of strength requirements and deformation limits. The deflection and rotations that are calculated in the AP model must be compared against the deformation limits that are specific to each component type. If any structural element or connection violates an acceptability criteria (strength or deformation), modifications must be made to the model before it is re-analyzed.

The ***acceptability criteria for flexural loads*** (Figure 6.5) is based on the flexural design strength of the structural element, including the strength reduction factor Φ, and the over-strength factor Ω applied to the material properties as appropriate.

Structural Behavior	Acceptability Criteria	Subsequent Action for Violation of Criteria
Element Flexure	ΦM_n[A]	Section 5-3.1.1
Element Combined Axial and Bending	AISC LRFD 2003 Chapter H Interaction Equations	Section 5-3.1.2
Element Shear	ΦV_n[A]	Section 5-3.1.3
Connections	Connection Design Strength[A]	Section 5-3.1.4
Deformation	Deformation Limits, defined in Table 5-3	Section 5-3.2

UFC 4-023-03, 2004

Component	AP for Low LOP		AP for Medium and High LOP	
	Ductility (μ)	Rotation, Degrees (θ)	Ductility (μ)	Rotation, Degrees (θ)
Beams--Seismic Section[A]	20	12	10	6
Beams--Compact Section[A]	5	-	3	-
Beams--Non-Compact Section[A]	1.2	-	1	-
Plates	40	12	20	6
Columns and Beam-Columns	3	-	2	-
Steel Frame Connections; Fully Restrained				
Welded Beam Flange or Coverplated (all types)[B]	-	2.0	-	1.5
Reduced Beam Section[B]	-	2.6	-	2
Steel Frame Connections; Partially Restrained				
Limit State governed by rivet shear or flexural yielding of plate, angle or T-section[B]	-	2.0	-	1.5
Limit State governed by high strength bolt shear, tension failure of rivet or bolt, or tension failure of plate, angle or T-section[B]	-	1.3	-	0.9

Figure 6.5
Acceptability Criteria and Deformation Limits for Steel Members (UFC Criteria)

When the internal moment (flexural required strength) determined by the AP model exceeds the flexural design strength of an element, the element is either removed or modified. For Linear Static models, structural elements that can sustain a constant moment while undergoing continued deformation must be modified through insertion of an effective plastic hinge. The designer must determine the location of the effective plastic hinge through engineering analysis and judgment or with the guidance provided for the particular construction type. In nonlinear static and dynamic models, the software must have the ability to adequately represent the nonlinear flexural response, after the internal moment reaches the flexural design strength of the element.

The acceptability criteria for ***columns undergoing combined axial loads and flexural loads*** is based on the strength criteria (force controlled case) derived from equations 5-10 and 5-11 and Table 5-6 of FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings".

If the ***shear design strength*** is exceeded, the member must be removed and the loads from that element must be redistributed.

If the design strength for any ***connection failure mode*** is exceeded, or if the allowable plastic rotation for a given connection is exceeded, the connection must be removed. If the connections at both ends of an element have failed, the loads from that element must be redistributed. Connection considerations for progressive collapse analysis are similar to those discussed previously as related to blast loads.

The overall ***GSA approach and performance criteria*** for nonlinear analysis are similar in most respects to the UFC approach. A second alternative is actually the primary recommended approach in the GSA Guidelines; a static "linear" approach. This approach is essentially identical to the "quasi-linear" *m*-factor approach presented in FEMA 356, sections 3.4.2.2 and as presented in Table 5-5. In the GSA guidelines, the *m*-factor is replaced with the term DCR, or demand-capacity ratio. The DCR, like the *m*-factor, is a multiplier for strength or capacity to account for or to allow some plasticity in deformation controlled actions. The simplicity of the DCR approach is that a progressive collapse load case can be formulated for use with standard design software that will permit direct design. Figure 6.6 presents the DCR values published in the GSA guidelines for steel beams and columns.

Component/Action	Values for Linear Procedures
	DCR
Beams-flexure	
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	3
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{ye}}}$	2
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.
Columns-flexure	
For 0<*P/P*$_{cl}$<0.5	
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	2
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \ge \frac{460}{\sqrt{F_{ye}}}$	1.25
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.

GSA Progressive Collapse Analysis and Design Guidelines, 2003

Figure 6.6
DCR Values from GSA Criteria for Beams and Columns

The DCR specified by the GSA guideline and discussed above is defined as:

$$DCR = \frac{Q_{ud}}{Q_{ce}};$$

where Q_{ud} is the acting force or demand on the component or connection, and Q_{ce} is the expected ultimate, un-factored capacity of the component or connection.

The DCR can be linearly equated with the combined static equivalent load case (described further in Section 6.4 below) and the factored member capacity. The GSA guidelines present their own table of strength increase factors. For typical steel shapes, this multiplier is 1.1. For the case of structural steel, the DCR for typical components is equal to 2.0 (it is less more several connections and columns).
Combining these factors then:

$$\frac{2(D+0.25L)}{\frac{1}{\phi}\left(1.1\langle \phi P_n, \phi M_n, \phi V_n\rangle\right)} \le 2.0;$$

where the ϕ factors applied in the design software are removed from the analysis, since, unlike the UFC criteria, the GSA guidelines do not require strength reductions. This results in:

$$D + 0.25L \le \frac{1.1}{\phi}\phi R_n;$$

and if ϕ is conservatively taken as 0.9,

$$\frac{D+0.25L}{1.22} \le \phi R_n,$$

which results in a load case for progressive collapse mitigation approximately and conservatively equal to:

$$0.82D + 0.2L$$

This load case can be used to “design” a structure for progressive collapse by applying it to models/cases where columns are removed per the guidance provided above. This is a conservative approach in that the design software will select members that will ensure that the DCR of 2.0 is not exceeded by this load case. The GSA procedures actually allow this value to be exceeded and a “hinge” inserted at the location in the component where this value is exceeded, as long as a three hinge mechanism (member failure) does not occur to an extent that the damaged area/volume criteria is exceeded. Thus, this approach is conservative in that no hinges are formed.

It should be noted that DCRs for steel are variable and material “overstrength” factors are not constant. Generally, DCR’s for steel members are a function of section compactness and type of moment connection. In fact, as Figure 6.7 from the GSA Guidelines illustrates, the DCR for connections will most often control the design or analysis.

Component/Action	Values for Linear Procedures DCR
Columns-flexure	
For <P/P_{cl}>0.5	
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{260}{\sqrt{F_{ye}}}$	1
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \geq \frac{400}{\sqrt{F_{ye}}}$	1
Columns Panel Zone-Shear	2
Column Core-Concentrated Forces	1.5
Fully Restrained Moment Connections	
Pre-Northridge (Pre 1995)	
Welded unreinforced flange (WUF)	2
Welded flange plate (WFP)	2
Welded cover plated flanges	2
Bolted flange plate (BFP)	2
Post Northridge (FEMA 350) Public Domain	
Improved WUF-bolted web	2
Improved WUF-welded web	2
Free flange	2
Welded top and bottom haunches	2
Reduced beam section	2
Post Northridge (FEMA 350) Proprietary	
Proprietary system	≤3
Partially Restrained Moment Connections	
Top and bottom clip angle	
a. Shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Tension failure of horizontal leg of angle	1.5
c. Tension failure of rivets or bolts	1.5
d. Flexural failure of angle	3
Double split tee	
a. Shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Tension failure of rivets or bolts	1.5
c. Tension failure of split tee stem	1.5
d. Flexural failure of split tee	3
Bolted flange plate	
a. Failure in net section of flange plate or shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Weld failure or tension failure on gross section of plate	1.5
Bolted end plate	
a. Yield of end plate	3
b. Yield of rivets or bolts	2 (rivets); 1.5 (high strength bolts)
c. Failure of weld	1.5
Composite top and clip angle bottom	
a. Failure of deck reinforcement	2
b. Local flange yielding and web crippling of column	3
c. Yield of bottom flange angle	3
d. Tensile yield of rivets or bolts at column flange	1.5 (rivets); 1 (high strength bolts)
e. Shear yield of beam flange connections	2
Shear connection with or without slab	2

GSA Progressive Collapse Analysis and Design Guidelines, 2003

Figure 6.7
DCR Values from GSA Criteria for Columns (cont'd) and Connections

6.4. What are the accepted static and dynamic analysis approaches?

The alternate path analysis approach requires that a three-dimensional structural system analysis be conducted, either using static equivalent loads or a dynamic analysis.

The load cases required by the GSA and UFC are as follows:

- For static equivalent analysis using the GSA criteria:

  $2\ (D + 0.25L)$

- For static equivalent analysis using the UFC criteria:

$2\ [\ (0.9 \text{ or } 1.2)\ D + (0.5\ L \text{ or } 0.2\ S)\ ] + 0.2\ W$

- For dynamic analysis using the GSA criteria:

  $(D + 0.25L)$

- For dynamic analysis using the UFC criteria:

$[\ (0.9 \text{ or } 1.2)\ D + (0.5\ L \text{ or } 0.2\ S)\ ] + 0.2\ W$

where:
D = Dead load (kN/m^2 or lb/ft^2)
L = Live load (kN/m^2 or lb/ft^2)
S = Snow load (kN/m^2 or lb/ft^2)
W = Wind load, as defined for the Main Wind Force-Resisting System in Section 6 of ASCE 7-02, (kN/m^2 or lb/ft^2)

The 0.9 or 1.2 factor in the UFC equations is derived from section C2.5, "Load Combinations for Extraordinary Events", which provides an identical equation for "checking a structure to determine its residual load carrying capacity following the occurrence of a damaging extraordinary event", where "selected load bearing elements should be notionally removed and the capacity of the remaining structure evaluated…"[xx]

The factor of 2.0 in both the GSA and UFC static equivalent equations is intended to account for the load amplification due to inertial effects. This load should only be applied over the bays (on a floors) adjacent to the notionally removed member, as shown in Figure 6.8.

Preliminary research[xxi] has shown that this factor of 2.0 is very conservative. Thus, a dynamic nonlinear analysis approach will yield a considerably more efficient design.

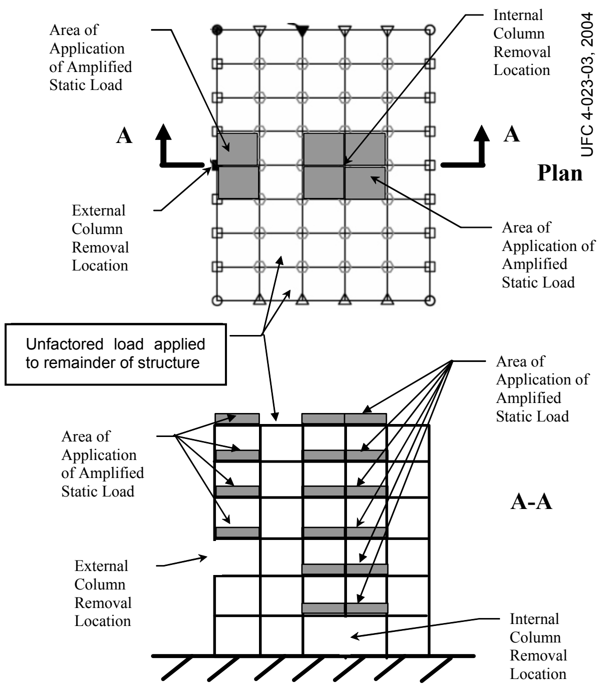


Figure 6.8
UFC Guidance for Static Equivalent Load Application for Alternate Path Analysis

Loads from failed elements must be accounted for in the analyses if the floor area criteria are not already exceeded. The UFC criteria recommend that, for a nonlinear dynamic analysis, the designer:

- double the loads from the failed element to account for impact and apply them instantaneously to the section of the structure directly below the failed element, and

- apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

For a linear or nonlinear static analysis, the loads on the failed element are already doubled, then:

- the loads from the failed element are applied to the section of the structure directly below the failed element, or

- if the loads on the failed element are not doubled, then double them and apply them to the section of the structure directly below the failed element, and

- apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

6.5. What are the specific steps involved in an analysis for progressive collapse mitigation?

While blast analysis of steel components and structures generally involved a single dynamic nonlinear approach, three options exist for progressive collapse analysis per the existing guidelines: the static "quasi-linear" GSA DCR approach, a static nonlinear frame or system analysis or a dynamic nonlinear frame or system analysis. The dynamic nonlinear analysis approach is generally recommended, since the static equivalent approach tends to over compensate for inertial loads in both the GSA and UFC load formulations. This approach can be tedious depending on the software interface available and the complexity of the structure. The GSA static nonlinear analysis is not recommended, since existing nonlinear deformation limit criteria (as presented in the 2003 Guidelines) are more restrictive than the GSA linear approach. Thus, if GSA criteria are applicable, use either the GSA static linear (more conservative, less efficient) or the dynamic nonlinear (less conservative, more efficient) approach.

The following steps are generally involved in performing a ***dynamic nonlinear alternate path analysis for a new design***:

- Design the structural system in accordance with applicable codes and standards (Figure 6.9),

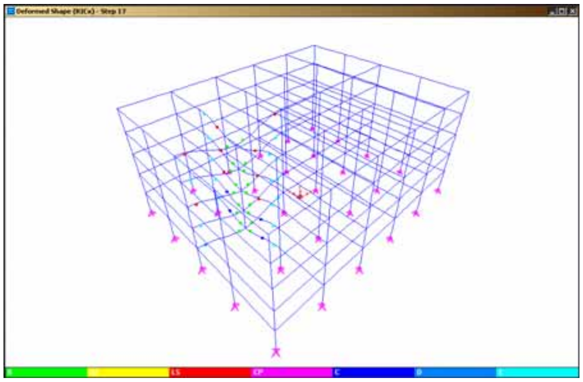


Figure 6.9
Simple 4x5 Bay, Example 5 Story Structure

- Either through "micro" analysis, or using test data or established criteria (UFC or GSA member rotation allowables, connection plastic rotation allowables, etc.) prepare the input for nonlinear hinge definition in the structural model (note that member hinges, column hinges and connection hinges will have different nonlinear properties) (Figure 6.10),

- Define and apply the progressive collapse load case to the structural model,

- Per the requirements of either the GSA or UFC criteria, remove the vertical component from the model, retaining the forces acting at that interface,

- Remove those forces from the model linearly in time over a period short enough such that removal does not influence calculated displacements. (Cursory investigation has shown[xxii] that a time not longer than about one-twentieth of the natural period of the structural system will be satisfactory for many structures),

- Evaluate the dynamic deformations of the structure and the resulting hinge rotations to determine whether deformation allowables have been exceeded in members or connections. If so, redesign members to reduce rotations, (Figure 6.11)

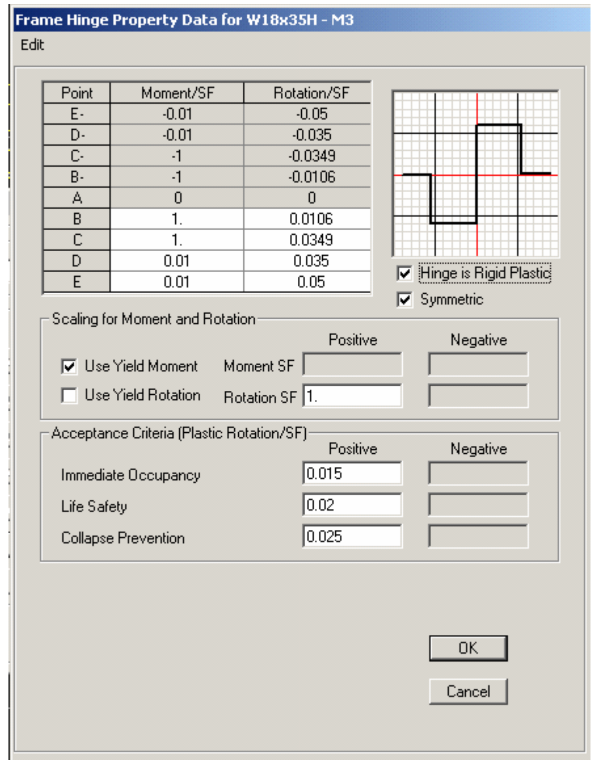


Figure 6.10
Nonlinear Hinge Definition for Alternate Path Analysis

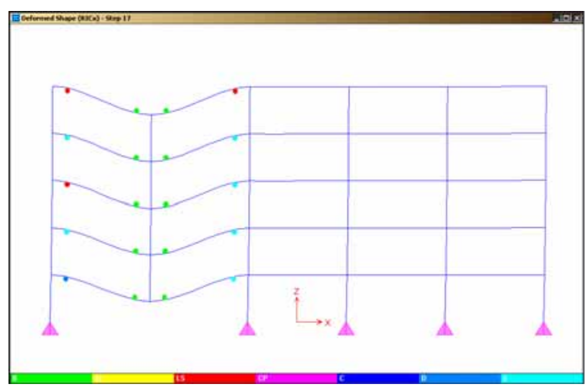


Figure 6.11
Nonlinear Hinge Rotations in Initial Alternate Path Analysis

- Evaluate force controlled mechanisms (shear, column moment interaction, member axial capacity), and increase member size if exceeded. (Note that axial-moment interaction in nonlinear analysis should be evaluated per the guidelines of FEMA 356, Table 5-6, where rotation limits are a function of axial load to axial capacity ratios),

- Revaluate structure until allowable deformations and forces are satisfied with member and connection capacities.

Studies[xxiii] of fairly simple 5 by 4 bay, 5-story steel moment resisting frame structures suggest that the dynamic analysis approach is significantly more efficient. Designing in accordance with "best practice" lateral drift requirements, the existing design "passes" the alternate path analysis with no changes.

6.6. What are the best approaches for improving the performance of the structure given the loss of a key component?

The best approach for progressive collapse mitigation is likely a combination of the indirect methods for providing redundancy, by providing additional flexural capacity and ductility by ***satisfying the alternate path analysis criteria***, and by providing additional global structural continuity by ***satisfying tie force requirements*** presented in the current guidelines. It must be emphasized that the alternate path approach is not meant to satisfy a specific threat scenario for single column removal, but is simply an indication that a measurable capacity for progressive collapse resistance exists.

The DoD UFC has additional prescriptive requirements that will add to this resistance to progressive collapse. These include the requirement that:

- "For all Levels of Protection, all ***multistory vertical load carrying elements*** must be capable of supporting the vertical load after the loss of lateral support at any floor level (i.e., a laterally unsupported length equal to two stories must be used in the design or analysis)."[xxiv] For this analysis, the UFC requires that the unfactored (no 2.0 factor applied for inertial effects) load case be used, and that the appropriate strength reduction factors and over-strength factors be applied. The loads from the "removed" story need not be applied to the wall or column.

- "In each bay and at all floors and the roof, the slab/floor system must be able to withstand a net upward load of the following magnitude:

$$1.0\,D + 0.5\,L$$

where:

D = Dead load based on self-weight only (kN/m^2 or lb/ft^2)

L = Live load (kN/m^2 or lb/ft^2)"

(Note that this load is applied to each bay, one at a time, i.e., the uplift loads are not applied concurrently to all bays. The floor system in each bay and its connections to the beams, girders, columns, capitals, etc, must be designed to carry this load. A load path from the slab to the foundation for this upward load does not need to be defined. The dynamic increase factor of 2.0 is not applied to this load case.)

- Finally, the UFC requires that all perimeter columns be designed with sufficient shear capacity such that full plastic flexural moment can be developed.

6.7. What types of retrofit or techniques are available for key components?

Steel structures that are determined to have a high potential for progressive collapse can be strengthened effectively and economically. Effective strengthening concepts can include global strengthening of the steel frame system by creating a multilevel Vierendeel truss, and/or local upgrading of steel beam-to-column connections to increase structural integrity, ductility, and beam-to-beam continuity.

Alternatively, when specific threat information is available, the direct design alternative is "key element design" or the provision of "specific local resistance." For progressive collapse mitigation, this generally equates to column hardening and transfer element hardening. For example, in a structure where the loss of one perimeter column may not compromise the structural system in terms of local collapse, the loss of two columns may. Thus, all perimeter columns may be specifically designed to resist the application of a large vehicle bomb one-half bay distance away, thus providing sufficient column capacity to prevent the loss of more than a single column for that specific threat.

The best current approaches for steel column hardening are concrete encasement, replacement with concrete-filled box sections, HSS or pipe,, and base plate protection through embedment of the column-to-base plate connection into the slab.

[xix] "Design of Buildings to Resist Progressive Collapse," Unified Facilities Criteria UFC 4-023-03, Department of Defense, approved for public release, distribution unlimited, July 2004, Chapter 5, Structural Steel Design Requirements, 5-2.

[xx] "Minimum Design Loads for Buildings and Other Structures," SEI/ASCE 7-02, 2003, Section C2.5, Load Combinations for Extraordinary Events, page 243.

[xxi] Preliminary analysis on 5x4 bay 5 story steel structures with 25-ft typical bay spans for internal Walter P. Moore "Best Practices" documents has suggested that inertial "multipliers" of between 1.3 and 1.5 are more realistic when members achieve significant plastic rotation/deformations. These multipliers were developed by comparing average and total (additive) hinge rotations in nonlinear static and nonlinear dynamic SAP V8 models used in the analysis.

[xxii] As determined in the Walter P. Moore analyses described above.

[xxiii] Crowder, B., Stevens, D.J. and Marchand, K.A., "Design of Buildings to Resist Progressive Collapse," short course proceedings, Security Engineering Workshop sponsored by the Virginia Society of Professional Engineers Tidewater Chapter and the DoD Security Engineering Working Group; US Army Corp of Engineers, Naval Facilities Engineering Command, Air Force-Civil Engineering Support Agency, June 2004, Progressive Collapse Mitigation Example Problems: Session 1: Tie Force Calculations and Alternate Path Analysis for Steel Frame Structures, slide 64.

[xxiv] UFC 4-023-03, Chapter 2, Progressive Collapse Design Requirements for New and Existing Construction, page 2-6.

SECTION 7
A RECENT HISTORY OF BLAST AND COLLAPSE EVENTS

7.1. What have we learned from recent terrorist attacks, including both World Trade Center attacks and the Pentagon attacks?

It is likely true that past structures were able to resist abnormal extraordinary loads due to inherent overstrength and continuity, and that more contemporary structures, taking advantage of recent developments in optimization, innovative framing systems, and refinement of analysis techniques have resulted in structures with a considerably smaller margin of safety. It may also be true that framing systems designed for ease of construction possess less inherent continuity leading to less resistance to abnormal loads (i.e., less load redistribution).[xxv]

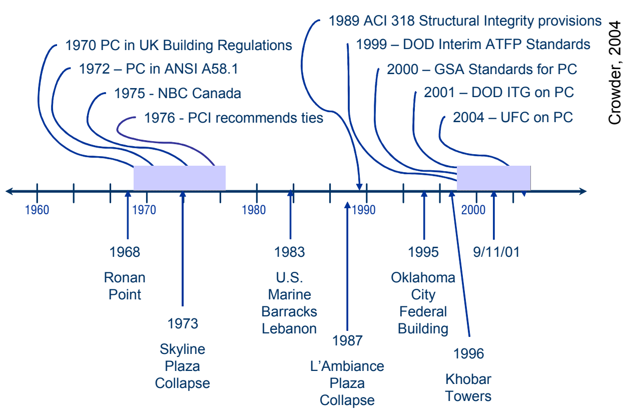


Figure 7.1
Progressive Collapse Event Timeline—1968-2001

Figure 7.1 presents a timeline of many progressive collapse events worldwide and design and engineering responses to those events. While design methods have steadily improved over the last 35 years, these gradual changes in construction processes and details, combined with the advent of increased terrorist attacks, have brought building designers to a point in time where robust and functional designs for blast and progressive collapse mitigation may be prudent. Unfortunately, still, only limited guidance exists in current standards, guidelines and codes. Research is being accomplished and guidelines developed, but our experience with recent attacks can provide some of the best qualitative guidance we might find.

Survey and assessment team reports for the Murrah Building bombing (Oklahoma City), the 1993 World Trade Center bombing, the 2001 World Trade Center attacks and the 2001 Pentagon attack summarize damage, observations and lessons learned from these tragic events. (Figure 7.2)

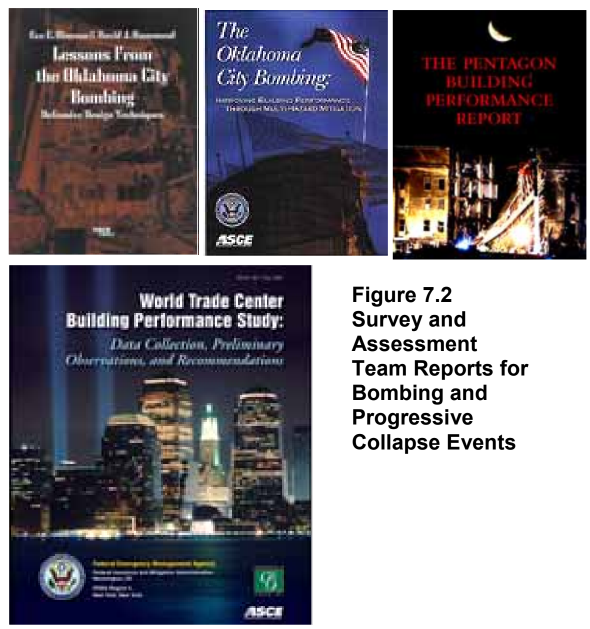


Figure 7.2
Survey and Assessment Team Reports for Bombing and Progressive Collapse Events

The attacks on the World Trade Center towers in September of 2001 illustrated inherent robustness and reserve capacity in the towers themselves and in several buildings surrounding the site. Ironically, the events on that day show progressive collapse was adverted in many cases. The towers themselves were able to withstand the initial loading from the impact of the airplanes, only to succumb at a later time due to the additional loading/weaknesses created due to the fire-structure interaction. In addition, several buildings surrounding the towers received extraordinary damage from debris loading, but they did not exhibit general collapse, i.e. they did not experience progressive collapse. Examples of response in the Towers, the Banker's Trust Building, and in World Financial Center 3, American Express-New York, are further described below:

- ***World Trade Center 1 and 2 – New York***—The highly redundant steel exterior moment frame was able to bridge about 140 ft of missing columns[xxvi] (Figure 7.3)

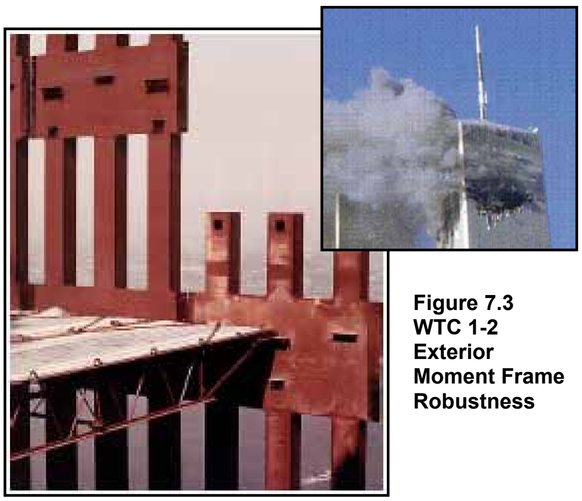


Figure 7.3
WTC 1-2
Exterior
Moment Frame
Robustness

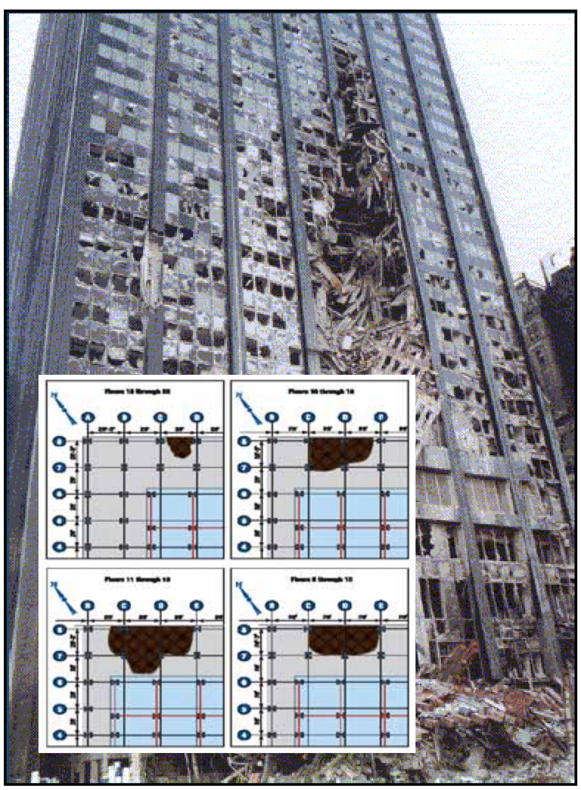


Figure 7.4
Bankers Trust Building, New York

- ***Bankers Trust – New York***—Debris from collapse of WTC 2 removed an exterior column over a partial height of the building. The redundancy of the structure provided the necessary bridging to transfer loads from the missing column. (Figure 7.4)

- ***World Financial Center 3, American Express – New York***—Sections of the corner column were destroyed. The corner bay was supported by cantilevered structure above and stiffening provided by the exterior wall system. (Figure 7.5)

Figure 7.5
World Financial Center 3, American Express—New York

- ***World Trade Center 7 – New York***—
This structure survived seven hours of continuous and widespread fires. Collapse of the structure was finally initiated by failures at the transfer trusses on floors 5-7.

The attack on the Pentagon also offers some useful information regarding the performance of this older and heavier structure. In ASCE's "The Pentagon Building Performance Report", the task group found that several key factors contributed to the mitigation of extensive collapse:[xxvii]

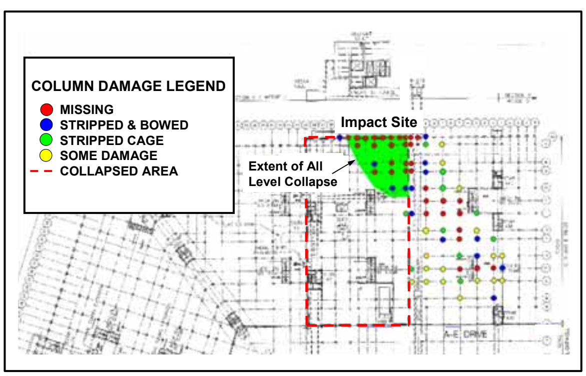
- Redundant and alternative load paths of the beam and girder framing system
- Short spans between columns

- Substantial continuity of beam and girder bottom reinforcement through supports
- Design for unreduced 150 psf warehouse live load in excess of service load
- Significant residual load capacity of damaged spirally reinforced columns
- Ability of exterior walls to act as transfer girders

These authors also cited several measures that were "validated" through the performance of the structure:

- Continuity, as in the extension of bottom beam reinforcement through the columns
- Redundancy, as in the two-way beam and girder system
- Energy-absorbing capacity, as in the spirally reinforced columns
- Reserve strength, as provided by the original design for live load in excess of service

Figure 7.6 illustrates locations and severity of column damage in the Pentagon.



Crowder, 2004

Figure 7.6
Plan Showing Pentagon Column Damage

7.2. Does the history of deliberate attacks within and outside the U.S. teach us anything about the current capability of our structures to resist these loads?

Other attacks around the world have illustrated that some of the provisions extant outside the U.S. and being considered for implementation in U.S. codes may have, in fact, been proven successful in at least two events. The British tie force provisions were instituted into the U.K. design codes and made law in 1970 after the Ronan Point apartments progressive collapse in London in 1968. These provisions (ties and bridging) were incorporated into all commercial and government buildings from that time forward. Two IRA bombings since that time are worthy of review, as they suggest that tying in particular may provide observable benefits.

Exchequer Court, St. Mary's Axe, City of London (Financial District), 1992—SEMTEX bomb explosion 6 m from building. Exchequer Court was a "modern" steel frame building with cast-in-place concrete floors and braced frame construction, so all connections were designed for shear only, except that the design had to provide tying forces in accordance with British standards. The connections were flush-end plate type shear connections (PR). The building was significantly damaged but did not suffer collapse. Figure 7.7 shows damage extent and steel column ductility observed in the St. Mary's Axe event of 1992.

Crowder, 2004

Figure 7.7
Exchequer Court, St. Mary's Axe, City of London

Kansallis House, Bishopsgate, City of London 1993—Constructed in the early 1980's of cast-in-place reinforced concrete, eight stories high with reconstituted stone wall cladding. The perimeter beam was a 575mm deep by 300mm wide reinforced concrete beam. A grid of reinforced concrete columns supported each floor and along the side of the building nearest the explosion the columns were 1200mm wide and 300mm deep. The bomb exploded within 6 meters of building. Three load bearing columns and 127m^2 of the 1st floor and 73m^2 of the 2nd and 3rd floors immediately above these columns were lost. In spite of this damage the majority of the building remained intact. The mechanism that enabled the remaining part of the building to bridge over the missing columns is unknown but clearly the design provisions provided sufficient redundancy in the structure to allow such bridging to occur.

Figure 7.8 shows the street façade damage and failed columns of Kansallis House.

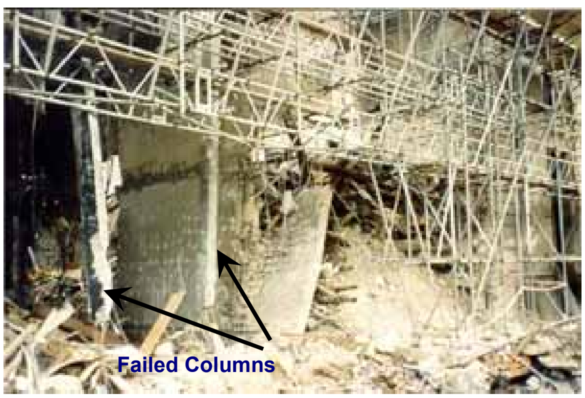


Figure 7.8
Kansallas House, City of London

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xxv Crowder, B., Stevens, D.J. and Marchand, K.A., "Design of Buildings to Resist Progressive Collapse," short course proceedings, Security Engineering Workshop sponsored by the Virginia Society of Professional Engineers Tidewater Chapter and the DoD Security Engineering Working Group; US Army Corp of Engineers, Naval Facilities Engineering Command, Air Force-Civil Engineering Support Agency, June 2004, Progressive Collapse – Historical Perspective, slide 4.

xxvi Magnusson, John, "Learning from Structures Subjected to Loads Extremely Beyond Design," presented at the 2003 AISC and Steel Institute of New York Steel Building Symposium: Blast and Progressive Collapse Resistance (symposium proceedings), American Institute of Steel Construction, 2004, page 6.

xxvii Mlaker, Paul F., et al, "The Pentagon Building Performance Report," American Society of Civil Engineers (ASCE), the Structural Engineering Institute (SEI), January 2003, page 58.

SECTION 8
RESEARCH AND FUTURE NEEDS

8.1. Is there new guidance being developed?

The new UFC 4-023-03, "Design of Buildings to Resist Progressive Collapse" will complement existing GSA guidance for progressive collapse. The companion and follow-on document to this AISC Facts publication will be a "Design Guide for Blast and Progressive Collapse", to be published by AISC. ASCE currently also has two efforts underway, including a consensus blast design standard and an updated "State-of-the-Practice Report. Thus, new and more detailed guidance is being developed, and is being supplemented by current and planned research.

8.2. What are the main steel structure response issues that are still undetermined?

Key issues that remain unresolved concerning progressive collapse mitigation and the performance of steel connections under high blast demands include:

- The specific mechanics by which a moment resisting frame devolves from a flexure dominant system to a tensile membrane or catenary dominant system, and what are the rotation demands on connections at this devolution point,
- The reserve axial tension capacity of steel beam-to-column connections (i.e., "simple" and moment-resisting) after reaching significant inelastic rotations,
- The importance and impact of analysis approaches chosen; e.g., is a static linear alternate path analysis predictably conservative or unreliable?
- The overall effectiveness of progressive collapse mitigation provisions for buildings subjected to "real" threats; e.g., what does redundancy "buy you" with respect to a suicide bomber at the column perimeter, for a vehicle bomb at the curb, for a tanker truck across the street?
- The effects of blast loads on beam-to-column connection performance including severe beam and column twist, lateral bending, and strain rate effects on weld and base material ductility.

8.3. What kind of research is ongoing or planned for the near future?

Several current research initiatives are progressing [sponsored by the Defense Threat Reduction Agency (DTRA), the GSA, and the Technical Support Working Group (TSWG)] to investigate "key" issues related to the response of steel structures to blast loads and progressive collapse mitigation in steel structures.[xxviii] These include:

- Tests and design recommendations for baseplate configurations and designs to resist direct shear failure at column bases
- Tests and design recommendations for steel splice configurations subjected to blast loads
- Tests and design recommendations for box sections, HSS and steel pipe and concrete-filled sections subjected to direct blast loads
- Tests and evaluation of connections under direct blast loads
- Determination of post-blast gravity load-carrying capacity of a double span beam following column removal

[xxviii] Crawford, John, et al., “Test Planning Studies Pursuant to Developing Test Program for Components of Steel Frame Buildings,” presentation at “Steel Frame Structures Kickoff Meeting” for the Defense Threat Reduction Agency (DTRA), Albuquerque, NM, July 2004.

ORGANIZATIONS AND ACRONYMS

ACI	American Concrete Institute
AISI	American Iron and Steel Institute
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
CFD	Computational Fluid Dynamics
COE	Corps of Engineers
CUREe	Consortium of Universities for Research in Earthquake Engineering
DCR	Demand-Capacity Ratio
DoD	Department of Defense
DoS	Department of State
DTRA	Defense Threat Reduction Agency
FEA	Finite Element Analysis
FEMA	Federal Emergency Management Agency
GSA	General Services Administration
IBC	International Building Code
ISC	Interagency Security Committee
LRFD	Load and Resistance Factor Design
NAVFAC	Naval Facilities Command
NIBS	National Institute of Building Sciences
NIST	National Institute of Standards and Technology
OCA	Office of the Chief Architect (of the GSA)
P-i	Pressure-impulse
SAC	SEAOC-ATC-CUREe Joint Venture
SDOF	Single-Degree-of-Freedom
SEAOC	Structural Engineers Association of California
SEI	Structural Engineering Institute
UBC	Uniform Building Code
UFC	Unified Facilities Criteria
USACE	US Army Corps of Engineers
WTC	World Trade Center

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