PROGRESSIVE COLLAPSE BASICS

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ABSTRACT

Progressive collapse is the collapse of all or a large part of a structure precipitated by damage or failure of a relatively small part of it. The phenomenon is of particular concern since progressive collapse is often (though not always) disproportionate, i.e., the collapse is out of proportion to the event that triggers it. Thus, in structures susceptible to progressive collapse, small events can have catastrophic consequences.

After the progressive and disproportionate collapse of the Ronan Point apartment tower in England in 1968, prevention of progressive collapse became one of the unchallenged imperatives in structural engineering, and code-writing bodies and governmental user agencies attempted to develop design guidelines and criteria that would reduce or eliminate the susceptibility of buildings to this form of failure. These efforts tended to focus on improving redundancy and alternate load paths, to ensure that loss of any single component would not lead to a general collapse. But in fact, redundancy is only one of the ways of reducing susceptibility to disproportionate collapse. Improved local resistance for critical components and improved continuity and interconnection throughout the structure (which can improve both redundancy and local resistance) can be more effective than increased redundancy in many instances. Through an appropriate combination of improved redundancy, local resistance and interconnection, it should be possible to greatly reduce the susceptibility of buildings to disproportionate collapse.

INTRODUCTION

On the morning of 16 May 1968, Mrs. Ivy Hodge, a tenant on the 18th floor of the 22-story Ronan Point apartment tower in Newham, east London, struck a match in her kitchen. The match set off a gas explosion that knocked out load-bearing precast concrete panels near the corner of the building. The loss of support at the 18th floor caused the floors above to collapse. The impact of these collapsing floors set off a chain reaction of collapses all the way to the ground. The ultimate result can be seen in Figure 1: the corner bay of the building has collapsed from top to bottom. Mrs. Hodge survived but four others died.

Fig. 1. Ronan Point building after 16 May 1968 collapse

While the failure of the Ronan Point structure was not one of the larger building disasters of recent years, it was particularly shocking in that the magnitude of the collapse was completely out of proportion to the triggering event. This type of sequential, one-thing-leading-to-another failure was labeled "progressive collapse" and the engineering community and public regulatory agencies resolved to change the practice of building design to prevent the recurrence of such tragedies.

PROGRESSIVE COLLAPSE AND DISPROPORTIONATE COLLAPSE

Progressive collapse can be defined as collapse of all or a large part of a structure precipitated by failure or damage of a relatively small part of it. The General Services Administration (GSA, 2003b) offers a somewhat more specific description of the phenomenon: "Progressive collapse is a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse."

It has also been suggested that the degree of "progressivity" in a collapse be defined as the ratio of total collapsed area or volume to the area or volume damaged or destroyed directly by the triggering event. In the case of the Ronan Point collapse, this ratio was of the order of 20.

By any definition, the Ronan Point disaster would qualify as a progressive collapse. In addition to being progressive, the Ronan Point collapse was *disproportionate*. A corner of a 22-story building collapsed over its entire height as a result of a fairly modest explosion, an explosion that did not take the life of a person within a few feet of it. The scale of the collapse was clearly disproportionate to the cause.

While the Ronan Point collapse was clearly both progressive and disproportionate, it is instructive to examine other collapses in the same light.

Murrah Federal Office Building

The Murrah Federal Office Building in Oklahoma City was destroyed by a bomb on 19 April 1995. The bomb, in a truck at the base of the building, destroyed or badly damaged three columns. Loss of support from these columns led to failure of a transfer girder. Failure of the transfer girder caused the collapse of columns supported by the girder and floor areas supported by those columns. The result was the general collapse evident in Figure 2.

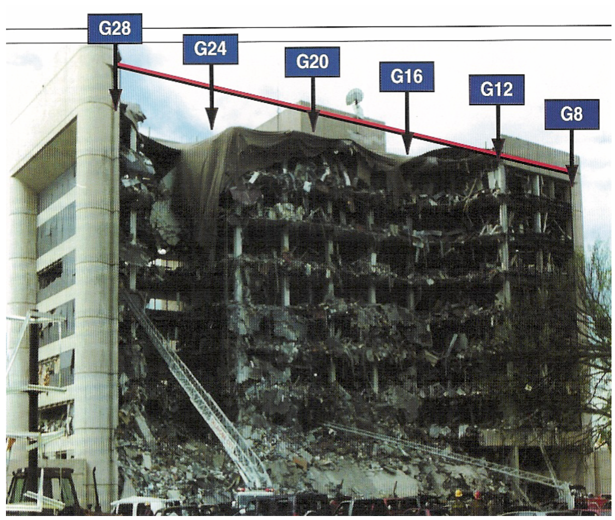

Fig. 2. Murrah Federal Office Building after 19 April 1995 attack

The Murrah Building disaster clearly was a progressive collapse by all the definitions of that term. Collapse of a large part of the building was precipitated by destruction of a small part of it (a few columns). The collapse also involved a clear sequence or progression of events: column destruction; transfer girder failure; collapse of structure above.

But was the Murrah Building collapse disproportional? The answer is not nearly as clear as in the case of the Ronan Point collapse. The Murrah collapse was large. But the cause of the collapse, the bomb, was very large too, large enough to cause damage over an area of several city blocks.

Ultimately, we must judge the Murrah Building collapse "possibly disproportional" only in the sense that we know now that with some fairly modest changes in the structural design (as will be discussed), the damage from the bomb might have been significantly reduced.

World Trade Center 1 and 2

Each of the twin towers of World Trade Center 1 and 2 collapsed on 11 September 2001 following this sequence of events: A Boeing 767 jetliner crashed into the tower at high speed; the crash caused structural damage at and near the point of impact and also set off an intense fire within the building (see Fig. 3); the structure near the impact zone lost its ability to support the load above it as a result of some combination of impact damage and fire damage; the structure above collapsed, having lost its support; the weight and impact of the collapsing upper part of the tower caused a progression of failures extending downward all the way to the ground.

Fig. 3. World Trade Center 1 and 2 on 11 September 2001

Clearly, this was a "progressive collapse" by any definition. But it cannot be labeled a "disproportionate collapse." It was a very large collapse caused by a very large impact and fire. And unlike the case with the Murrah Building, simple changes in the structural design that might have greatly reduced the scale of the collapse have not yet been identified.

Observations on "Progressive" and "Disproportionate" Collapse

Prevention of progressive collapse is generally acknowledged to be an imperative in structural engineering today. But in fact, virtually all collapses could be regarded as "progressive" in one way or another, and a building's susceptibility to progressive collapse should be of particular concern only if the collapse is also disproportionate. Indeed, the engineering imperative should be not the prevention of progressive collapse but the prevention of disproportionate collapse (be it progressive or not).

CODES AND STANDARDS

Since the progressive collapse of the Ronan Point apartment tower in 1968, many codes and standards have attempted to address the issue of this type of collapse. A complete survey of these efforts is beyond the scope of this paper, but a small sampling of current and recent provisions related to progressive collapse will provide an indication of the alternative approaches being considered and the direction in which these efforts appear to be evolving.

ASCE 7-02

The American Society of Civil Engineers *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2002) has a section on "general structural integrity" that reads thus: "Buildings and other structures shall 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. This shall be achieved through an arrangement of the structural elements that provides stability to the entire structural system by transferring loads from any locally damaged region to adjacent regions capable of resisting those loads without collapse. This shall be accomplished by providing sufficient continuity, redundancy, or energy-dissipating capacity (ductility), or a combination thereof, in the members of the structure."

Clearly, the focus in the ASCE standard is on redundancy and alternate load paths over all other means of avoiding susceptibility to disproportionate collapse. But the degree of redundancy is not specified, and the requirements are entirely threat-independent.

ACI 318-02

The American Concrete Institute *Building Code Requirements for Structural Concrete* (ACI, 2002) include extensive "Requirements for structural integrity" in the chapter on reinforcing steel details. Though the Commentary states that it "is the intent of this section … to improve …

redundancy" there is no explicit mention of redundancy or alternate load paths in the Code. The Code provisions include a general statement that "In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure" and many specific prescriptive requirements for continuity of reinforcing steel and interconnection of components. There are additional requirements for the tying together of precast structural components. None of the ACI provisions are threat-specific in any way.

GSA PBS Facilities Standards 2000

The 2000 edition of the GSA's *Facilities Standards for the Public Buildings Service* (GSA, 2000) included the following statement under the "Progressive Collapse" heading in the "Structural Considerations" section: "The structure must be able to sustain local damage without destabilizing the whole structure. The failure of a beam, slab, or column shall not result in failure of the structural system below, above, or in adjacent bays. In the case of column failure, damage in the beams and girders above the column shall be limited to large deflections. Collapse of floors or roofs must not be permitted."

This is an absolute and unequivocal requirement for one-member (beam, slab, or column) redundancy, unrelated to the degree of vulnerability of the member or the level of threat to the structure.

GSA PBS Facilities Standards 2003

The 2003 edition of the GSA's *Facilities Standards for the Public Buildings Service* (GSA, 2003a) retained the "Progressive Collapse" heading from the 2000 edition, but replaced all of the words reproduced above with this short statement: "Refer to Chapter 8: Security Design."

The structural provisions in Chapter 8 apply only to buildings deemed to be at risk of blast attack. For such buildings, the chapter provides general performance guidelines and references to various technical manuals for study of blast effects. This represents a complete change of approach from the 2000 version of the same document.

GSA Progressive Collapse Guidelines 2003

The GSA *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects* (GSA, 2003b) begins with a process for determining whether a building is exempt from progressive collapse considerations. Exemption is based on the type and size of the structure (for instance, any building of over ten stories is non-exempt) and is unrelated to the level of threat. Typical non-exempt buildings in steel or concrete have to be shown by analysis to be able to tolerate removal of one column or one 30-ft length of bearing wall without collapse. Considerable detail is provided regarding the features of the analysis and the acceptance criteria.

In some ways, these guidelines appear to be a throw-back to the GSA's PBS Facilities Standards of 2000 in that their central provision is a requirement for one-member redundancy, unrelated to the degree of vulnerability of the member or the level of threat to the structure.

METHODS OF PREVENTING DISPROPORTIONATE COLLAPSE

There are, in general, three alternative approaches to designing structures to reduce their susceptibility to disproportionate collapse:

- Redundancy or alternate load paths
- Local resistance
- Interconnection or continuity

Redundancy or Alternate Load Paths

In this approach, the structure is designed such that if any one component fails, alternate paths are available for the load in that component and a general collapse does not occur. This approach has the benefit of simplicity and directness. In its most common application, design for redundancy requires that a building structure be able to tolerate loss of any one column without collapse. This is an objective, easily-understood performance requirement.

The problem with the redundancy approach, as typically practiced, is that it does not account for differences in vulnerability. Clearly, one-column redundancy when each column is a W8x35 does not provide the same level of safety as when each column is a 2000 lb/ft built-up section. Indeed, an explosion that could take out the 2000 lb/ft column would likely destroy several of the W8 columns, making one-column redundancy inadequate to prevent collapse in that case. And yet, codes and standards that mandate redundancy do not distinguish between the two situations; they treat every column as equally likely to be destroyed.

In fact, since it is generally much easier to design for redundancy of a small and lightly-loaded column, redundancy requirements may have the unfortunate consequence of encouraging designs with many small (and vulnerable) columns rather than fewer larger columns. For safety against deliberate attacks (as opposed to random accidents), this may be a step in the wrong direction.

Local Resistance

In this approach, susceptibility to progressive/disproportionate collapse is reduced by providing critical components that might be subject to attack with additional resistance to such attacks. This requires some knowledge of the nature of potential attacks. And it is very difficult to codify in a simple and objective way.

Interconnection or Continuity

This is, strictly speaking, not a third approach separate from redundancy and local resistance, but a means of improving either redundancy or local resistance (or both). Studies of many recent building collapses have shown that the failure could have been avoided or at least reduced in scale, at fairly small additional cost, if structural components had been interconnected more effectively. This is the basis of the "structural integrity" requirements in the ACI 318 specification (ACI, 2002).

Approaches Used in Codes and Standards

The following tabulation shows which of these approaches to preventing disproportionate collapse are used in each of the five codes and standards discussed previously. Redundancy is the clear favorite, being the primary approach used in three of the five sources. [The rational threat-dependent analysis specified in the 2003 GSA PBS Facilities Standard could include any or all of the three design approaches.]

Approaches for design against disproportionate collapse adopted in selected codes and standards	Redundancy	Local Resistance	Interconnection	Threat-dependent analysis
ASCE 7-02	●			
ACI 318-02			●	
GSA…PBS, 2000	●			
GSA…PBS, 2003				●
GSA PC Guidelines	●			

CASE OUTLINES

To illustrate the techniques for reducing susceptibility to disproportionate collapse, consider how redundancy, local resistance or interconnection might have been used to improve the performance of Ronan Point, the Murrah Building and WTC 1 and 2.

Case Outline -- Ronan Point

Greater redundancy would have been difficult to build into the type of structure employed in the Ronan Point tower. Improved local resistance, in the form of greater strength of the precast concrete wall panels that blew out, precipitating the collapse, would not have helped; the panels would have blown out regardless of their strength. Better interconnection of structural components is the key for this structure. Stronger and more positive connections between the wall panels and the floors, with less reliance on friction due to weight to hold everything together, is likely to have greatly reduced the scale of the collapse of the Ronan Point building.

Case Outline – Murrah Building

The columns at the front face of this reinforced concrete building were at 20-ft centers on upper floors and 40-ft centers at ground level, with a transfer girder to make the transition. A requirement for one-column redundancy would almost certainly have eliminated the transfer: The smaller columns 20 ft apart would have extended down to the ground and the structure would have been designed to tolerate the loss of one of them. Would this have reduced the magnitude of the collapse on 19 April 1995? Probably not. The explosion would almost certainly have taken out several (at least five) of the small closely-spaced columns, easily overwhelming the one-column redundancy built into the design, leading to a collapse not significantly different from what actually occurred.

Improved local resistance, within plausible limits, would not have prevented destruction of the ground-floor column closest to the bomb. But improved ductility and shear capacity of the columns (possibly through the use of the kind of reinforcing steel details used in earthquake-prone regions), and better interconnection and continuity throughout the building, could have prevented the loss of any of the other large ground-floor columns and could have limited the collapse to a 60- to 80-ft width of structure from the ground to the roof — a major disaster but much less than what actually happened. The conclusion, then, is that the performance of the Murrah building on 19 April 1995 would not have been improved by a requirement for redundancy in the design, but could have been improved by better interconnection and continuity throughout the structure and different reinforcing steel details in the columns.

Case Outline – WTC 1 and 2

The exterior frame of each WTC tower was already so highly redundant that greater redundancy would be hard to contemplate. The interior columns were not redundant, except for the limited redundancy created by the hat trusses. But the impact and fire damage were so pervasive that greater redundancy in the interior is not likely to have changed the outcome. Greater local resistance (in the strictly structural sense, fire protection may be a different issue) was not a practical proposition for these towers. Finally, notwithstanding early reports to the contrary, connection failures do not now appear to have contributed significantly to the disaster, so improved interconnection would not have been useful.

The conclusion that none of the typical means of preventing disproportionate collapse would have been useful for the WTC towers reinforces the idea that the collapse of these buildings was not disproportionate to begin with.

Application of Codes and Standards to the Cases Considered

A tabulation showing which of the approaches to preventing disproportionate collapse were used in each of five selected codes and standards was presented earlier. That tabulation is expanded below to show whether use of those codes and standards in the design of Ronan Point, the Murrah Building and the WTC towers would plausibly have improved the performance of those structures. The results (see the box in the tabulation) indicate that use of current codes and user-agency standards would not consistently provide assurance against the types of collapse that occurred in those buildings — not even against the clearly disproportionate collapse at Ronan Point or the "possibly disproportionate" collapse at the Murrah Building.

Would use of these codes and standards in their design have improved the performance of Ronan Point, Murrah and WTC?	Redundancy	Local Resistance	Interconnection	Threat-dependent analysis	Ronan Point	Murrah Building	WTC 1 & 2
ASCE 7-02	●				?	N	N
ACI 318-02			●		Y	?	N
GSA…PBS, 2000	●				?	N	N
GSA…PBS, 2003				●	N	Y	N
GSA PC Guidelines	●				N	N	N

Y=yes; N=no; ?=maybe

SUMMARY AND CONCLUSIONS

Progressive collapse is the collapse of all or a large part of a structure precipitated by damage or failure of a relatively small part of it. Prevention of progressive collapse is one of the unchallenged imperatives in structural engineering today. But in fact, a building's susceptibility to progressive collapse should be of particular concern only if the collapse is also disproportionate. Indeed, the engineering imperative should be not the prevention of progressive collapse but the prevention of disproportionate collapse (be it progressive or not).

There are, in general, three approaches to designing structures to reduce their susceptibility to disproportionate collapse:

- Redundancy or alternate load paths, where the structure is designed such that if any one component fails, alternate paths are available for the load in that component and a general collapse does not occur

- Local resistance, where susceptibility to progressive/disproportionate collapse is reduced by providing critical components that might be subject to attack with additional resistance to such attacks

- Interconnection or continuity, which is, strictly speaking, not a third approach separate from redundancy and local resistance, but a means of improving redundancy or local resistance or both

The emphasis on redundancy over all alternatives in some recent codes and standards and user-agency requirements may not lead to buildings that are less susceptible to disproportionate collapse as a result of deliberate attack.

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BLAST BASICS

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ABSTRACT

This article (a primer on blast effects) begins by identifying general military and most commonly used commercial explosives. The manner in which explosives release their energy is described and the primary blast parameters for explosions in air are identified. The emphasis is on blast from surface explosions. This is followed by a brief discussion of the interaction of the blast wave with building structures. Strength of buildings when subjected to blast effects of high yield (nuclear) explosions is quantified. This is followed by a brief discussion of internal explosions. The final topic is a brief presentation of results of studies dealing with casualties in buildings produced by external blast.

INTRODUCTION

The purpose of this article is as a primer on the basic aspects of explosion phenomena as this applies to the effects of high explosives (military and commercial) and effects of accidental explosions from commercial substances such as natural gas, propane, and liquid fuels, etc., on structures and people.

EXPLOSIVES

Deliberate explosives come in two general categories, i.e., military and civilian or commercial. Military explosives include cased explosives such as bombs, mortar shells, bullets, etc., each designed for a specific form of delivery. This category also includes uncased explosives such as various plastic explosives used for demolition and other functions. These are referred to as high explosives. Low explosives include such products as propellants.

Commercial explosives include such products as dynamite, TNT (trinitrotoluene) and Ammonium Nitrate among others. Ammonium Nitrate is an essential ingredient in nearly all commercial explosives. Its predominant use is in the form of AN prill, a small porous pellet with fuel oil. More than two million pounds of these mixtures, commonly referred to as ANFO (Ammonium Nitrate Fuel Oil), are consumed each year. They account for approximately 80% of the domestic commercial market.

ANFO products have found extensive use in a variety of blasting applications including surface mining of coal, metal mining, quarrying and construction. Their popularity has increased because of economy and convenience. The most widely used ANFO product is oxygen balanced free-flowing mixture of about 94% ammonium nitrate prills and 6% No. 2 Diesel fuel oil.

Items which are capable of exploding, but whose primary function is not to act as explosives, include natural gas, propane, liquid fuels such as gasoline and many other chemicals. These are generally referred to as low explosives (Longinow, A., Alfawakhiri, F., 2003)

EXPLOSIONS

High explosives release their energy by a process called detonation, and low explosives, such as propellants, natural gas, propane, etc., by the process of rapid burning. The time required for the detonation of a quantity of high explosive is much less than that for the burning of a like amount of propellant. With high explosives, the rate of detonation is not markedly affected by the particle size; with propellants, grain size is all-important. The shattering effect of a high explosive detonation is great, that of low explosives much less so. These distinctions are not completely clear-cut, however.

A number of so-called low explosives can be made to detonate--even black powder, under great pressure, and proper conditions. The military (and terrorist) use to which high explosives are put depend on their great shattering power and their high rate of detonation.

Some high explosives, such as mercury fulminate for example, are very sensitive to heat and shock and can be easily detonated by a spark or other local application of heat. These types of explosives are used to initiate less sensitive explosives and are called primers. Other explosives less sensitive to heat and shock than primers are used as boosters, i.e., intermediates between the primer and the main body of the explosive. These are capable of being initiated by the former and of initiating the latter.

The quantities of these three types of explosives in a given weapon differ greatly. 1) A very small quantity of primer, usually less than one gram, is used; 2) the booster weight is ordinarily of the order of a pound to a few pounds; and 3) the bulk of the explosive content of a weapon, the insensitive part, may constitute over 99% of explosive.

The explosion of a booster gives rise to a compression wave in the main explosive. If detonation in the main explosive does not occur, this compression is propagated as a wave, at approximately the speed of sound, through the explosive. However, if the compression is sufficient, chemical reaction of the explosive will take place as the result of the elevated pressure and temperature in the compression wave. This chemical reaction is very rapid, and the products of the reaction have a very high pressure and temperature. This zone in which the chemical reaction takes place, called the detonation wave, is propagated through the explosive at a speed considerably in excess of the speed of sound in the explosive and is preceded by the compression wave which it supports.

The propagation velocity of the detonation wave depends on the chemical and physical properties of the explosive and, to some extent, on the dimensions of the explosive and the degree of confinement.

When the detonation wave reaches the interface between the explosive and the air that surrounds it (unconfined charge), the products of the detonation, largely gases, expand with high velocity, pressure and temperature. The boundary between the air and the hot compressed gases is sharply defined. Behind this layer the pressure and temperature at a short time interval later decrease rapidly to lower values toward the interior of the charge. The rate of expansion of the luminous zone (the burnt hot gases) continually decreases. Eventually another discontinuity emerges from the luminous zone and leaves it behind. This is the shock wave, a sharp discontinuous rise in pressure propagating through the air surrounding the explosion products.

If the charge is confined by a metal case, such as a steel case of a bomb, the pressure of the hot gases expands the case. At first the metal flows plastically, until the volume of the case has been increased considerably (about twofold for steel cases) and than it ruptures. The resulting fragments of the casing are propelled at a high speed, and since they are not at first retarded, they precede the shock wave over a great distance from the charge. The acceleration of the fragments requires energy, and the fragments may carry a considerable fraction of the detonation energy of the explosive away. For this reason, the energy and the pressure of the shock wave from a confined charge are considerably less than that from an uncased explosive charge (Longinow, A., Alfawakhiri, F., 2003; Glasstone S., Dolan, P. J., 1977)

PROPAGATION OF SHOCK WAVE IN AIR

As mentioned earlier, the rapid expansion of the mass of hot gases resulting from detonation of an explosive charge gives rise to a wave of compression called a shock wave, which is propagated through the air. The front of the shock wave can be considered infinitely steep, for all practical purposes. That is, the time required for compression of the undisturbed air ahead of the wave to the full pressure just behind the wave is practically zero.

If the explosive source is spherical, the resulting shock wave will be spherical, and since the surface is continually increasing, the energy per unit area continually decreases. As a result, as the shock wave travels outward from the charge, the pressure in the front of the wave, called the peak pressure, steadily decreases. At great distances from the charge, the peak pressure is infinitesimal, and the wave, therefore, may be treated as a sound wave.

Behind the shock wave front, the pressure in the wave decreases from its initial peak value. Near to the charge, the pressure in the tail of the wave is greater than that of the atmosphere. However, as the wave propagates outward from the charge, a rarefaction wave is formed which follows the shock wave. At some distance from the charge, the pressure behind the shock-wave front falls to a value below that of the atmosphere, and then rises again to a steady value equal to that of the atmosphere. The part of the shock wave in which the pressure is greater than that of the atmosphere is called the positive phase, and immediately following it, the part in which the pressure is less than that of the atmosphere is called the negative phase.

The pressure in the shock front and the pressure, temperature and composition of the undisturbed medium uniquely determines the speed at which the shock propagates. The greater the excess of peak pressure over that of the atmosphere, the greater the shock velocity. Since the pressure at the shock front is greater than that at any point behind it, the wave tends to lengthen as it travels away from the charge. In other words, the distance between the shock front and the part at which the pressure in the wave has decreased to atmospheric continually increases.

The time elapsing between the arrival of the shock front and the arrival of the part in which the pressure is exactly atmospheric is called the positive phase duration (see above). The quantity of interest in application of blast measurements is positive impulse, which is the area under the positive phase duration curve.

For each pressure range there is a particle or wind velocity associated with the blast wave that causes a dynamic pressure on objects in the path of the wave. In the free field, these dynamic pressures are essentially functions of the air density and particle velocity. For typical conditions, standard relationships have been established between the peak incident pressure, the peak dynamic pressure, the particle velocity, and the air density behind the shock front. The magnitude of the dynamic pressure, particle velocity, and the air density is solely a function of the peak incident pressure and, therefore, independent of the yield of the explosion. The following table (Table 1) lists particle velocities for overpressures in the range of 2 psi to 20 psi.

TABLE 1
OVERPRESSURES AND CORRESPONDING PARTICLE VELOCITIES

Overpressure (psi)	Velocity (feet/sec)	Velocity (miles/hour)
2	103	70
3	195	102
4	195	133
5	239	163
6	280	191
8	358	244
10	431	294
15	594	405
20	737	502

At very great distances from the charge, the wave becomes acoustic, i.e., the pressure rise, temperature rise, and particle velocity are all infinitesimal, and as mentioned above, the velocity of the wave is that of sound (Glasstone S., Dolan, P. J., 1977).

BLAST LOADS FROM SURFACE EXPLOSIONS

An explosion from a charge located on or very near the ground surface is referred to as a surface burst. The initial wave is reflected and reinforced by the ground surface to produce a reflected wave. The reflected wave merges with the incident wave at the point of detonation to form a single wave, essentially hemispherical in shape. A Mach front is formed by the intersection of the initial wave and the reflected wave from the ground. The shock front can be considered as a plane wave over the full height of the Mach front.

The height of the Mach front increases as the wave propagates away from the charge. The increase in height with distance is referred to as the path of the triple point and is formed by the intersection of the incident, reflected, and Mach waves. A structure is considered as subjected to a plane wave when the height of the triple exceeds the height of the structure (Glasstone, S., Dolan, P. J., 1977).

INTERACTION OF SHOCK WAVES WITH BUILDINGS

When a shock wave strikes a non-rigid obstacle such as a building, the wave is reflected by the surfaces of the building. At the instant the wave strikes the wall, the wall is accelerated and continues to accelerate as long as there is pressure on its outer surface. At first, the deformation of the wall is elastic, so that for insufficient excess pressure or insufficient positive duration there may be no permanent displacement of the wall. If the blast intensity is sufficient, the wall eventually deforms inelastically and suffers permanent displacement. If, for the wall in question, the displacement is greater than some critical amount, the wall will collapse.

The essential characteristics of loading and building response for transient loads produced by explosions depend primarily on the relationship between the effective duration of the loading and the fundamental period of the structure on which the loading acts. When the effective duration is very short, say less than one third of the period, then the impulse due to the transient loading is of major importance, and the response of the structure can be based entirely on a consideration of impulse and momentum. On the other hand, when the duration of the loading is relatively long compared with the fundamental period, then a quasi-static design can be made.

The effective duration of loading for a blast from one megaton of TNT is about 1 sec., for a kiloton equivalent energy about 0.1 sec., and for one ton, about 10 milliseconds. The duration varies as the cube of the energy of detonation. For energies corresponding to gas explosions or other explosives, the same relationship can be applied as a first approximation. The fundamental relations for developing blast-resistant design procedures are given in TM5-1300, 1990; TM5-855-1, 1986; Biggs, J. M., 1964; Newmark, N. M., 1953.

STRENGTH OF BUILDINGS

Depending on the geographic location, buildings are designed to resist gravity loads, wind loads, and seismic loads. Few, if any, conventional buildings are designed to resist blast loads.

During the decades of the cold war there was an interest in the United States to identify the best available shelter space. A fair amount of attention was devoted by the U.S. Civil Defense to determine the protective capabilities of existing buildings. The threat was that produced by the effects of nuclear weapons, which included thermal radiation, initial radiation and blast. Table 2 is a summary of a study conducted to categorize the strength of conventional buildings subjected to the effects of blast from nuclear weapons (Pickering, E. E., Bockholt, J. L., 1971)

TABLE 2
FAILURE OVERPRESSURES FOR CATEGORIES OF BUILDINGS

	Building type	**Free Field Failure Overpressure, psi**	**Failure includes either one or a combination of the following conditions**
1	Single story framed residences, with or without basements.	2.9	Roof collapse, gross displacement, collapse of walls.
2	Single story masonry load bearing residences with or without basements.	4.0	Same as "1"
3	Two or three story framed residences, row houses, motels, etc., with or without basements.	3.0	Roof collapse, gross displacement, collapse of walls, large portion of siding removed, gross deflection from vertical.
4	Two or three story masonry, load-bearing wall residences, apartments, motels, etc., with or without basements.	4.0	Same as "1"
5	One and two story "store front" and small commercial masonry load bearing wall buildings.	4.9	Same as "1"
6	Two to four story commercial, residential and office masonry load bearing wall buildings	5.0	Same as "1"
7	Multistory steel framed apartment buildings (two to ten stories). Heavy exterior walls - Light exterior walls -	 10.2 6.8	Exterior walls and interior partitions blown out. Severe distortion of frame. Severe distortion of interior core.
8	Multistory reinforced concrete frame apartment buildings, (four to ten stories). Heavy exterior walls - Light exterior walls -	 10.0 6.8	Same as "7"
9	Multistory steel framed office buildings (four to ten stories). Heavy exterior walls - Light exterior walls -	 10.4 8.0	Same as "7"
10	Multistory reinforced concrete framed office buildings (four to ten stories). Heavy exterior walls - Light exterior walls -	 10.2 8.0	Same as "7"

11	Steel framed office buildings, more than ten stories. Heavy exterior walls - Light exterior walls -	10.2 6.8	Same as "7"
12	Reinforced concrete framed office buildings. Heavy exterior walls - Light exterior walls -	12.0 6.5	Same as "7"
13	One story masonry load bearing wall buildings (such as schools, libraries, etc).	4.6	Same as "1"
14	Monumental masonry buildings, two to five stories, with or without structural frames	10.8	Roof collapse, gross displacement, collapse of exterior walls.
15	Masonry load bearing wall industrial buildings, one story	3.8	Same as "1"
16	Light steel frame industrial buildings, one story	6.4	Failure of structural frame.
17	Heavy steel frame industrial buildings, one story	10.0	Failure of structural frame.

BLAST LOADS FROM INTERNAL EXPLOSIONS

An internal or confined explosion will produce shock loads and, in most instances, quasi-static gas pressure loads from the confinement of the products of the explosion. This pressure has a long duration in comparison to that of the shock pressure.

As in free air, blast loads on a given surface will be generated from the direct shock wave and from shock waves reflected from other surfaces.

Even light partitions that fail in an explosion are present for a sufficient length of time (a few milliseconds) to provide reflecting surfaces for the shock wave and partial confinement of the gas loads.

Three degrees of confinement are possible, i.e., a) fully vented, b) partially confined, and c) fully confined. A fully vented explosion is one in which the gaseous products can escape through openings before significant gas pressure and impulse can be developed. A partially confined explosion is one in which venting does not occur quickly enough to eliminate gas pressure buildup, resulting in relatively long duration gas pressure loads and significant impulse. Full confinement is associated with essentially total confinement of an explosion as by a hardened structure. In this type of explosion, the gas pressure decays very slowly (NFPA 68, 2002).

Numerous deflagrations (explosions) on the inside of buildings are produced due to the accidental release, accumulation and subsequent ignition of gases such as natural gas, propane, etc.

CASUALTIES PRODUCED BY BLAST

People in buildings subjected to blast would be tumbled in the direction of the flow terminating in impact with hard surfaces inside the building. Some would interact with debris from the break up of the building walls, partitions, furniture, other people, etc. Some would be blown out of the building, terminating in impact with the ground plane. The extent of casualties would depend on the type of building, the yield of the weapon and the range of the given building from the weapon.

In a study (Longinow, A., 1979) conducted for U.S. Civil Defense, estimates of casualties were made for different buildings and shelters when subject to a large weapon (1 MT). For framed buildings (steel and concrete), up to four stories with weak exterior walls (weak curtain walls, large glass windows, etc.) the following results were obtained.

Probability of Survival	10%	50%	90%
Reference Free Field Overpressure, psi	11	7	5

Casualty mechanisms included blast translation terminating in impact with hard surfaces and interaction with debris from the breakup of the building walls, partitions, furniture, etc. These results are representative of large weapons in the Mach region and provide an indication of casualties that would be produced in buildings by external high explosives of smaller yields

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this narrative was to provide a very basic and general primer on explosions produced by surface bursts and their effects on buildings structures and people. The information on which this is based may be found in the open literature as shown in the list of references.

It is noted that a great deal of information dealing with the response of structures and people when subjected to blast effects comes from that developed for nuclear weapons during the decades of the cold war. This is not to say that such information is not useful in the present case. A lot of it is. Nonetheless, there is clearly a need to develop data that is specific to high explosives.

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LEARNING FROM STRUCTURES SUBJECTED TO LOADS EXTREMELY BEYOND DESIGN

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Jon Magnusson

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ABSTRACT

An important part of building structural design is strategically meeting certain performance objectives for a set of defined hazards. Understanding these concepts and communicating them to building owners, and even the general public, is becoming increasingly important.

Many buildings have been subjected to loads greatly in excess of their design criteria and have not collapsed. Lessons learned from several of these buildings are shared, including a "submarine" concept for building construction.

INTRODUCTION

Structures are designed for certain loads and hazards. Structural engineers need to communicate clearly with the building owner, architect, and building officials about what loadings may have been considered, or possibly more importantly, not considered in a project design.

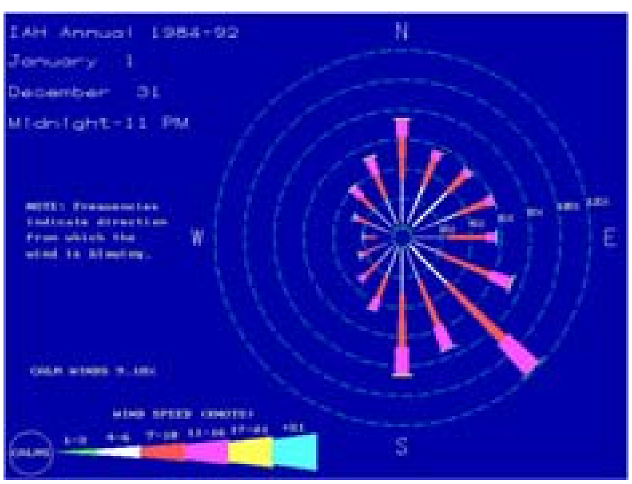
Many things can be learned from investigating structures that have been subjected to loads beyond what was contemplated in their design. The overloads may have been due to intentional malicious acts or accidental hazards. The damage patterns and behavior of members and connections can give hints into how to make structures more resistant to these overloads.

WHAT ARE THE OJBECTIVES OF "DESIGN"?

Building designers can not possibly design for every extremely remote hazard that their project may be subjected to in its life. Commercial buildings are not designed for meteorite impact, or for nuclear blasts, or for other kinds of military attacks. However, the design process does include looking at four major hazards:

1. Gravity
2. Wind
3. Earthquake
4. Fire

Each of these must be defined. Gravity is well-defined and extremely predictable! Fire is typically dealt with by mitigating the hazard through event control such as sprinklers, fireproofing, and active firefighting so that the structure does not need to take the fire load. Wind and earthquake are defined on a probabilistic basis that, while not as precise as gravity, is quite reliable. Figure 1 shows examples of this approach.



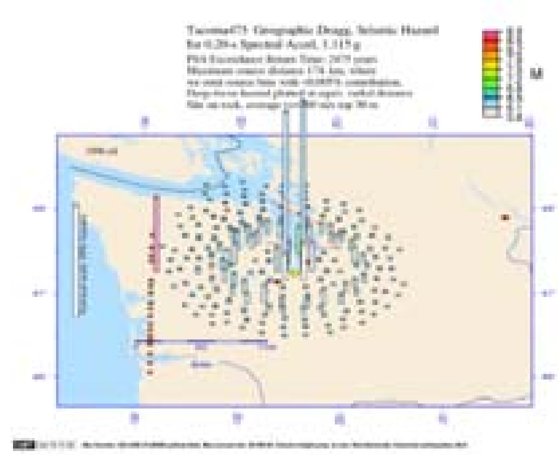


Fig. 1. Wind Speed and Direction Probabilities for Houston, TX and Seismic Hazard for Tacoma, WA

For each hazard, performance objectives are developed. Examples of performance objectives for wind and seismic are shown in Figure 2.

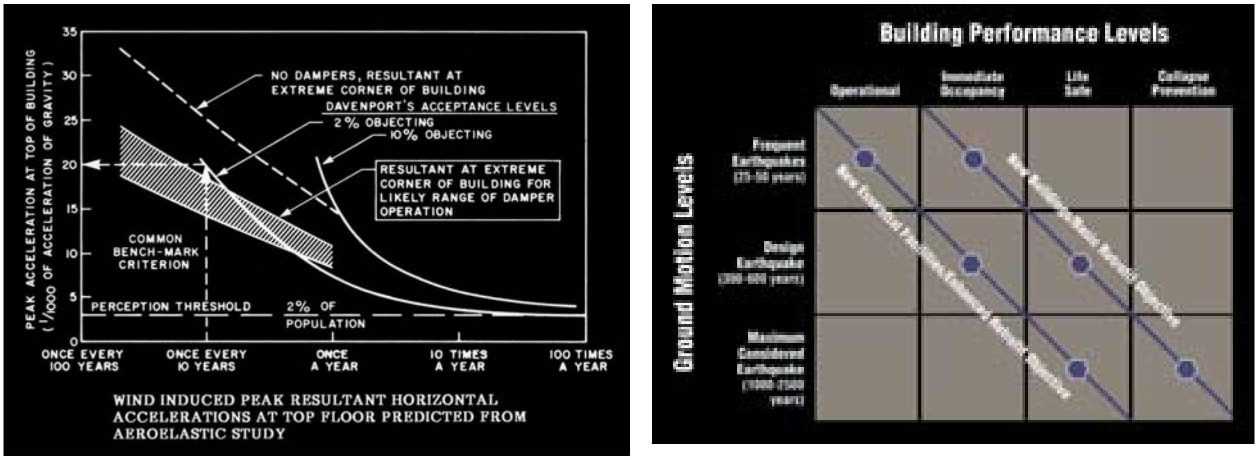


Fig. 2. Wind Acceleration and Seismic Ground Motion Performance Objectives

Once the design hazards and corresponding performance objectives are defined the design can proceed to bring these into conformance. For rational design, these steps must be repeated over and over for each element of the building system:

1. Hazard Definition
2. Performance Objectives
3. Conformance Strategies

It is critically important that all design disciplines have consistent performance objectives for the different design hazards. For example, if a sprinkler system is part of a conformance strategy for the structure, it had better have performance objectives that it be operational under the same hazard.

EXTREME LOADINGS BEYOND "DESIGN"

Usually the magnitude and probability of extreme loadings are not predictable. Unfortunately, many of the extreme loadings being considered now in designs are blast loadings due to intentional detonations intended by the perpetrators to cause damage and injury.

When the Murrah Federal Building in Oklahoma City was attacked the blast was equivalent to 4,000 lbs. of TNT. The hazard associated with a truck bomb could be 60,000 lbs. of TNT, or 15 times greater than the Murrah attack. And again, this is not an upper limit because it is always possible to postulate multiple trucks bombs in an attack.

The terrorists in the attack of September 11, 2001 ultimately had control of three planes (temporarily four) and could have used them all to attack one target. If "plane attacks" are to be considered as a design hazard, then much larger planes need to be considered. Figure 3 compares two planes that have already hit buildings, with two planes that are even larger.

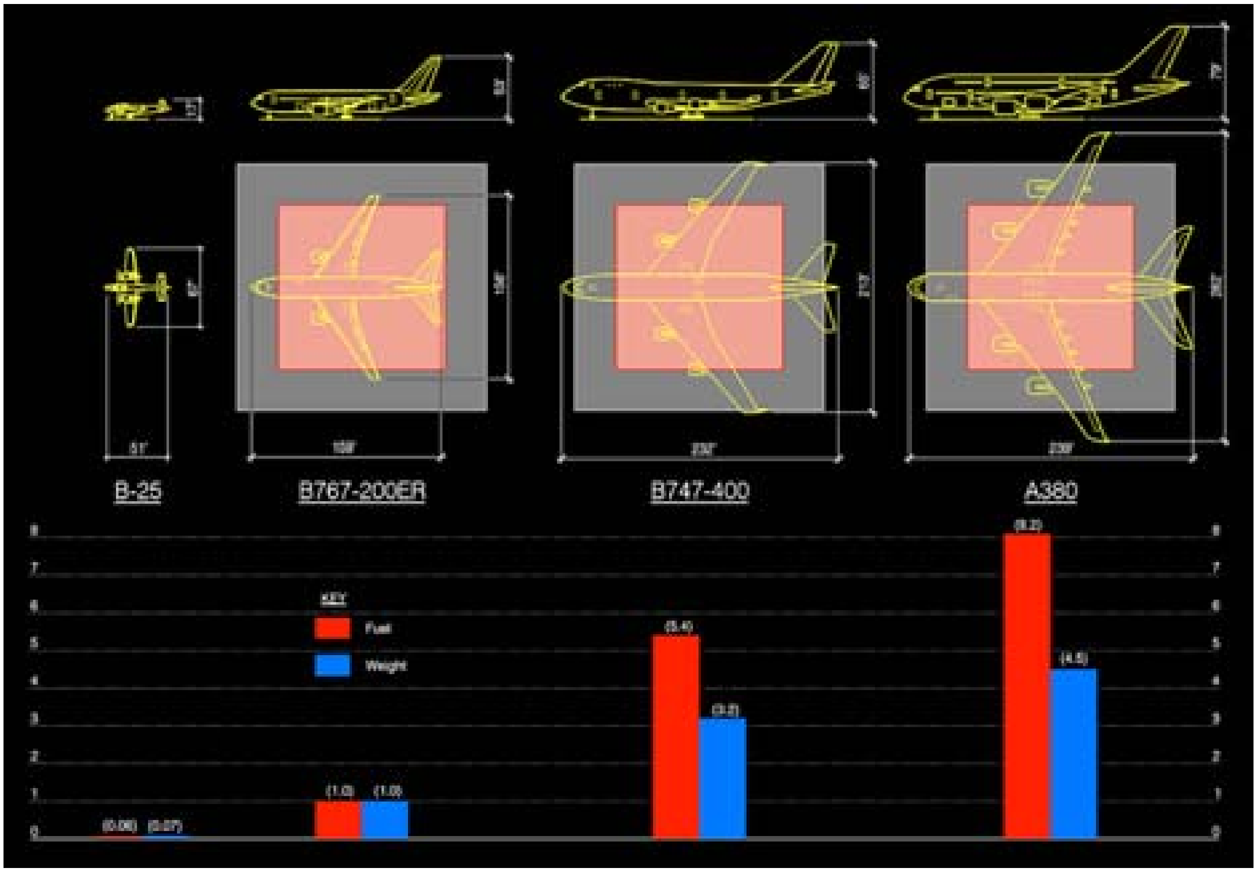


Fig. 3. B-25 hit Empire State Building, 767-200 used in WTC attacks.

Clearly, many of these hazards are beyond the realm of cost effective resistance, and in many cases beyond the ability to overcome the physics of the hazard.

COMMON SRUCTURAL STRATEGIES

One of the most common strategies to resist progressive collapse is to use a notional removal of one exterior element at a time and creating alternate load paths. This does not relate to any specific hazard and therefore does not create a performance objective for a "real" threat. It is simply meant to increase the redundancy of the structure. Many structures that have not been designed for this criterion actually have shown some capacity to lose a column without global collapse.

This approach generally results in much stronger horizontal framing systems with significant axial capacity. It is important to consider what happens when an unexpected hazard occurs that removes two or more columns. Does this strong horizontal construction then cause a horizontal propagation of the collapse? A New York City Fire Chief reported to the World Trade Center Building Performance Assessment Team that the structures that are most susceptible to progressive collapse are the ones that are well tied together. Mark Loizeaux of Controlled Demolition, Inc., whose occupation is taking down buildings, has also said that the easiest buildings to take down are the ones with high levels of continuity.

Designers should consider the possibility of negative impacts of excessive horizontal ties under more extreme loading when using the notional removal technique.

BUILDING CASE STUDIES

Ronan Point – United Kingdom

This is the most famous case of “pure” progressive collapse. There were five deaths. There was extensive vertical propagation of the collapse, but almost no horizontal propagation. If the building had been well tied together and the initiating event was larger, would the entire structure have collapsed?

Murrah Federal Building – Oklahoma City

Complete vertical and some horizontal propagation of the collapse. The blast was the equivalent of 4,000 lbs. of TNT.

600 California – San Francisco

Crane accident demonstrated tremendous ductility of concrete filled steel pipes.

World Trade Center 1 and 2 – New York

The highly redundant steel exterior moment frame was able to bridge about 140 feet of missing columns. Intense fires ultimately brought down both buildings.

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 above provided the necessary bridge to transfer loads from the missing column.

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.

World Trade Center 3, Marriott Hotel – New York

The Marriott was crushed by debris from both WTC 1 and WTC 2. WTC 2 hit it first and, even though hundreds of tons of debris partially collapsed the southern part of the building, the collapse did not propagate to the north. The floor connections were not strong enough to allow the propagation.

AN ALTERNATIVE STRATEGY

Based on observations of these buildings, the concept of structural compartments seems to have merit. Within each compartment, strong horizontal ties could be used to prevent vertical propagation of a collapse from a relatively small overload. In the event of a massive overload, the collapse would propagate horizontally until it hit an extra strong bulkhead wall (or one with weak connections) to arrest the collapse. This dual level protection concept is similar to the way that a submarine design deals with military hazards.

CONCLUSION

Regardless of the strategies employed it is critical to identify the design hazards, performance objectives, and conformance strategies and discuss these with the building owner, architect, and building officials so that all parties have appropriate expectations and understanding of risk.

CONVENTIONALLY DESIGNED BUILDINGS: BLAST AND PROGRESSIVE COLLAPSE RESISTANCE

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INTRODUCTION

The purpose of this paper is to review the inherent resistance of conventionally designed buildings for which specific blast or progressive collapse requirements have not been included in the design. Current blast and progressive collapse design guidelines are outlined and assessment of ordinary structural design vulnerabilities are discussed. Simple concepts are introduced to provide a more robust structure with respect to blast and progressive collapse threat.

BLAST AND PROGRESSIVE COLLAPSE

The loadings produced by blast events are typically much higher than the design loadings for which an ordinary structure is designed. These loadings, referred to as overpressures in the technical literature, have been extensively studied by the Department of Defense and other military agencies for their effect on various structures. Important parameters are the size of charge, distance of explosion to structure, ground or air burst, etc. The overpressures are usually measured in PSI – pounds per square inch on the element being impacted. This is noteworthy since most code design loads for ordinary structures are given in pounds per square foot. Relationships for the overpressure have been developed and given the size of the charge and distance from the structure a value can be calculated. As noted, these overpressures are usually well beyond the capacity of the structure. Local failures of structural elements in the region of the explosion is likely.

Since the risk or threat level is highly variable and local capacities are easily exceeded, more detailed analysis is unnecessary and it is commonly assumed the element impacted will fail. The effect of the blast is then studied by removing the impacted element (or elements) from the structure and then analyzing the modified structure.

The effect of the blast can be in the opposite direction for which the design loads were considered, resulting in existing capacities to resist blast being further reduced. Upward pressures from blast effects on the lower floors can put beams and girders into a reverse bending mode resulting in bottom flanges becoming compression elements with large unbraced lengths. The overpressures can easily overcome the downward design loads. The Oklahoma City Federal building is an example of the blast loading resulting in loading the concrete structure in a direction opposite to the main design resistance. Also, blast can produce significant side loading on elements that had no original design loads in that direction. Local floor failure can result in significant unbraced lengths on columns as witnessed in the first attack on the World Trade Center.

For ordinary buildings the best preventive measures are to keep the blast away from the structure with barriers, etc. (i.e. Defensive Design). The GSA (1) and DOD (2) have developed stand-off distances, barrier designs etc. for various threats. These documents address requirements for new and existing government buildings. ASCE has published a state of the art reference (3).

Progressive collapse is the disproportionate collapse of a structure due to a failure of a much smaller (albeit important) element. Obviously this includes but is not limited to blast effects. Since progressive collapse can encompass a much larger portion of the structure (or the entire structure) with many different collapse possibilities, a specific assessment approach is not possible. It is best to look at the specific guidelines outlined in the above documents and comment on what is missing in ordinary buildings to provide an evaluation of a design. Progressive collapse is a global assessment of the structure whereas blast is usually a local element assessment.

The GSA document provides an insight to current thinking related to mitigating progressive collapse. Basic to this approach is the concept of multiple load paths and structural redundancy which will produce a robust structure. Simply stated, the vulnerable element is removed and the structure should not collapse. The structure must have another load path to prevent collapse. Conditions from the structure which are considered "fundamental" by the GSA in new designs and upgrades are the "double span" design of beams and girders. Beams and girders need to be continuous through columns so that if a column is removed, the resulting structure can develop an alternate load path and carry the existing loads. Clearly most ordinary buildings could not meet this guideline even though a progressive collapse consideration does exist in some codes. In addition the GSA guideline recommends connection resilience similar to that developed with the AISC seismic standards for connections.

From a lateral system consideration the guidelines would develop designs with uniformly distributed moment frames on all the column grids as a first approach to robustness and redundancy. Bracing systems, which could be severely impacted by local blast effects are less robust than uniform moment frames and would be discouraged or combined with uniform moment frames. A perimeter moment frame strengthened on the first level above grade is also recommended.

As demonstrated clearly in the World Trade Center collapse, serviceability (wind stiffness) and redundancy can provide considerable reserve strengths for unexpected demands on a structure. The lateral system designed for wind and gravity had the strength and robustness to provide an alternative load path for a severely disrupted gravity system.

A brief list of some of the guidelines noted in the General Services Administration document is listed below:

- Continuity of floor members to produce alternate load paths.
- Redundancy for alternate load paths.
- Provide extra strength on first supported level above grade.
- Moment frames on the perimeter for protection of exterior elements. Keep girders same, oversize connections.
- Tie everything together – beneficial effects of composite construction.
- Provide extra reinforcing in first supported slab – to strengthen membrane effect.
- Consider longer effective lengths for first tier columns considering a local loss of floors.
- Consider loss of localized lateral system – bracing most vulnerable, uniformly distributed moment frames best.

EXISTING STRUCTURE - EVALUATION

After the events of a 9/11, many owners have requested a vulnerability analysis of their buildings. As an example of a structure not designed for any specific threat, a 39 story tower was investigated for the specific threat of losing one or two of its main vertical supports at the street level. Of interest here is not damage but collapse potential. The purpose of the analysis is not an "exact" simulation of a structural collapse, but to provide the analyst with useful information for assessing the performance of the structure and to make judgments on its safety. Capacity evaluation should be done without the usual factors of safety and member strength modifications.

The 39 story structure under consideration was comprised of typical structural steel construction with metal deck and concrete floors, steel beams and columns and several transfer girders at the lowest level. The lateral system is a fully welded perimeter moment frame of beams and columns. Because of transfers on several levels above the ground level the frame possessed additional strength in these levels. This is recommended in the guideline but came about because of gravity transfers.

An analysis of the structure was carried out incorporating the GSA load conditions of 2 (DL & .25L) with the columns removed. This analysis used a "push down" methodology (similar to the "push over" analysis) to capture the non-linear behavior of the perimeter frames as the full load was applied. Figures 1 through 4 show the results of analysis for two different locations of missing columns. The perimeter frame, originally controlled by wind drift design, indicated a capacity to redistribute the load to adjacent columns. As seen from the "push down " diagrams yielding did not begin until well above the original design load levels. An important consideration in this study was the investigation of the actual connection capacity to transfer the loads in the new load path. This analysis is approximate in that the membrane action of the slab is not considered which would provide additional resistance to the structure. More detailed programs such as RAM's PERFORM – COLLAPSE are now available which do include the important membrane effect.

CONCLUSIONS

By considering the recommended design guidelines of the current GSA progressive collapse and DOD Anti-Terrorist standards one can apply design standards to existing structures to evaluate their vulnerability. The following items are a brief summary of essential features.

Blast

- Defensive Design – Keep blast away from structure
- Element resistance – not available, capacity << demand, accept local failure
- Local analysis unnecessary – Remove element from structure and prevent progressive collapse

Progressive Collapse

- Alternate load paths, imperative
- Redundancy (goes hand in hand with above)
- Resilient connections.
- Over strength connections to create alternate load paths.
- Overall design continuity vs. local element resistance.

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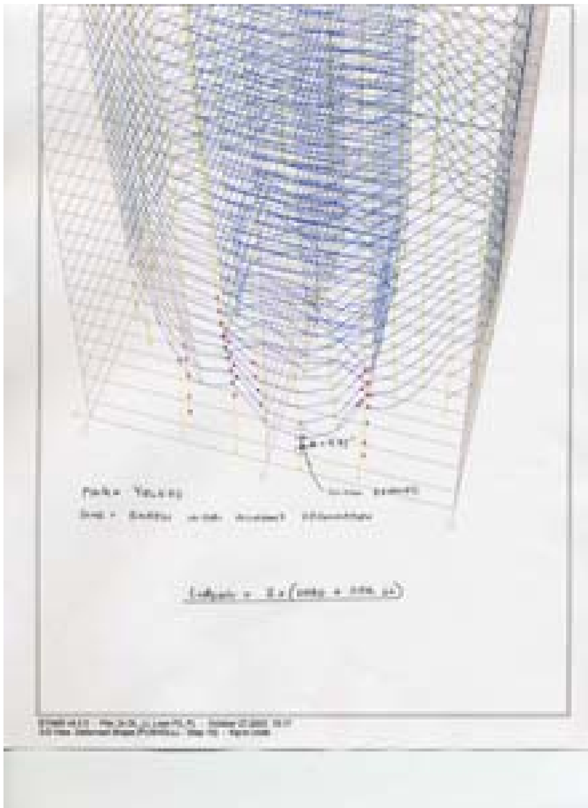

Figure 1

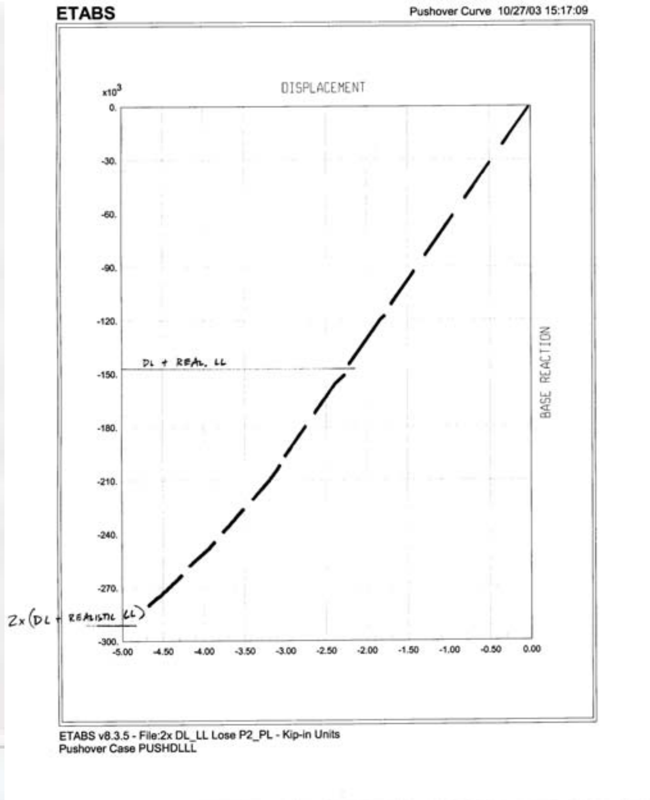

Figure 2

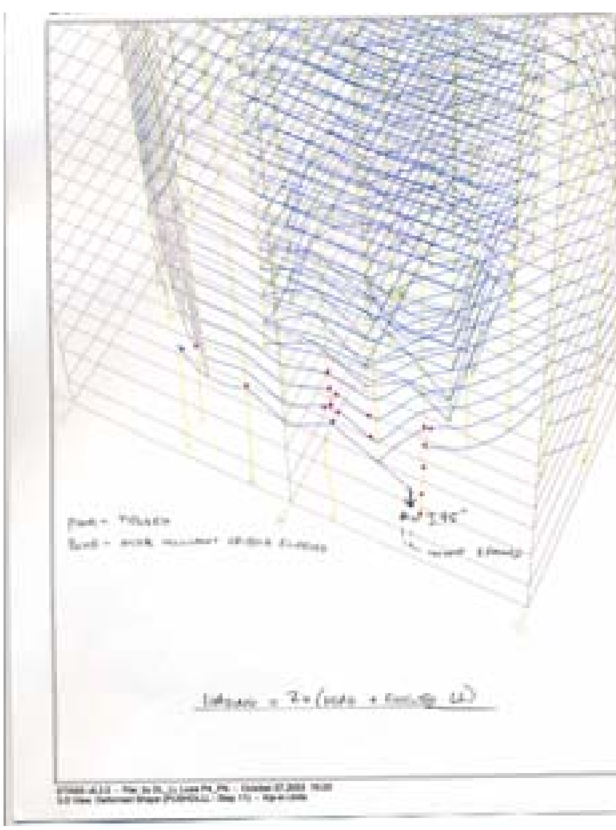

Figure 3

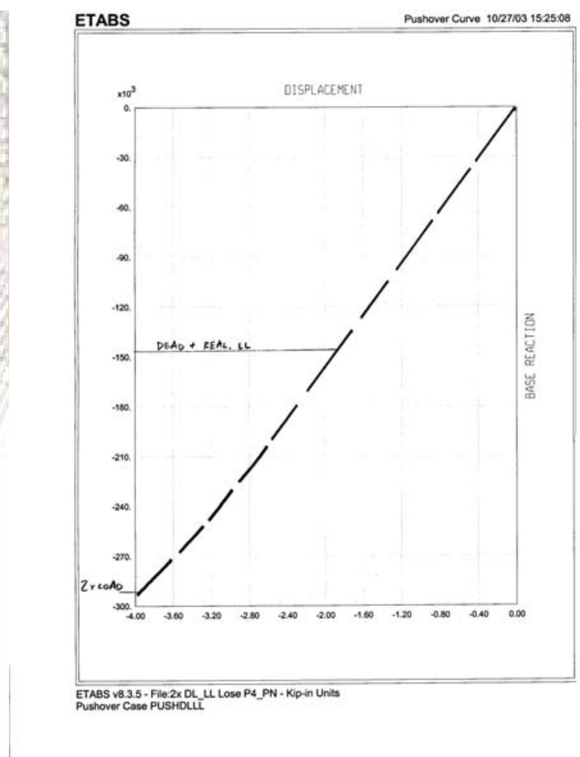

Figure 4

CONSIDERATIONS FOR RETROFIT OF EXISTING STEEL BUILDINGS FOR RESISTING BLAST AND PROGRESSIVE COLLAPSE

BY WILLIAM J. FASCHAN, RICHARD B. GARLOCK, AND DANIEL A. SESIL

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Richard B. Garlock P.E., M. S.

Mr. Sesil is a Partner and a Structural Engineer with twenty years of experience at LERA. He has worked on award-winning projects both in the United States and abroad, spanning the entire range of building types. His expertise in the design of structural systems for high profile, high security and public use facilities includes courthouses, embassies and banks for the Federal Reserve System. Mr. Sesil, also an Adjunct Assistant Professor at Columbia University, has completed numerous landmark projects of international renown including the William Jefferson Clinton Presidential Center, Rock & Roll Hall of Fame, Miho Museum & Bridge in Japan and the Prada SoHo boutique in New York City.

Daniel A. Sesil P.E., S.E., M. S.

ABSTRACT

Evaluation and subsequent strengthening of existing structures for extreme loading cases, such as blast, require a realistic and pragmatic design approach. Effective communication between Owner, Structural Engineer, Architect, Risk Analyst, Insurance Providers, and other stakeholders is paramount to a finished project that is satisfactory to all. The benefits of structural steel for use in the renovation of existing buildings are well documented and are applicable to the type of retrofitting required for resistance to blast and progressive collapse. The performance of steel construction during the 1993 bombing of The World Trade Center is further evidence. Combination of the existing conditions of the structure and the nature of the threat leads to strengthening techniques that may not be the first choice in the case of new construction. Less intrusive types of strengthening are favored. The general approach for strengthening of existing buildings starts with researching the original construction documents and then performing a condition assessment of the building. Vulnerability analysis is a multi-step process where there is constant dialogue about the possibilities of non-structural methods to decrease the threat on the building. With the goal of enhancing a building's performance under an extreme event, we have provided a range of upgrades from enhanced perimeter protection to structural hardening.

INTRODUCTION

Balancing cost and vulnerability is a challenge for the security of any new building. Add considerations of an existing structure, building occupants, decades-old design criteria, and you have begun the first step in the evaluation of the hardening prospects for an existing building. Today, many building owners who are developing new buildings with a high risk profile are considering extreme loading criteria for their building designs. The pool of existing buildings, however, is much greater, and owners of these buildings are questioning how their structures would perform under similar criteria.

Evaluation and any subsequent strengthening of existing structures for extreme loading cases, such as blast, require a realistic and pragmatic design approach. Effective communication between Owner, Structural Engineer, Architect, Risk Analyst, Insurance Providers, and other stakeholders is paramount to a successful finished project. With the goal of enhancing a building's performance under an extreme event, we have provided a range of upgrades from enhanced perimeter protection to structural hardening. The benefits of structural steel for use in the renovation of existing buildings are well documented and are applicable to the type of retrofitting required for resistance to blast and progressive collapse. This paper discusses the aspects of blast hardening specific to existing facilities. Lessons learned from the 1993 bombing of The World Trade Center are described. Following a discussion on a general approach for the hardening of existing structures, we discuss common goals and situations. Due to the confidential nature of the work, project names and identifying photos are not provided.

ASPECTS OF BLAST HARDENING SPECIFIC TO EXISTING FACILITIES

Effectively protecting an existing facility by blast hardening is a relatively difficult task. Realistically, the built environment has a number of inherent weaknesses when considering the possible effects of an extreme event. Rare is the facility that has systems designed for improved performance in an extreme event. Cladding, site planning, stairs, power systems, and structures are planned to deal with more common environmental conditions. Structures are typically constructed without specific consideration of redundancy or robustness in an extreme event.

While risk analysis and vulnerability assessment are essential first steps in any security project, these steps take on a special importance for an existing facility. Due to the particular difficulties of effectively hardening an existing building, it is important that the risk analysis and vulnerability assessment result in a clear understanding by the client of the potential vulnerabilities and of the scale of construction work that may be required to mitigate or prevent damage from the identified threats.

Since the costs of hardening an entire existing facility are often so high, clients commonly choose to focus their efforts on specific locations or functions within a facility where risks are highest. They establish limited hardening objectives. Frequently, non-structural security measures prove to be the client's most practical and cost-effective alternative. Preventing or re-routing pedestrian or vehicular traffic, instituting operational changes, providing redundancy in the building's critical power and sprinkler systems and other similar measures are often the most effective techniques for enhancing the performance of an existing facility.

Perhaps the single most important aspect of existing building security projects is the identification of practical alternatives, the best of which may not involve structural hardening.

Where a decision is made to harden some part of an existing facility or a specific structural system or element, the design approach is influenced by a series of factors that include the following:

- Information about relevant existing conditions is often limited;
- Structure to be renovated is commonly hidden or obstructed by existing architectural or building services systems that are difficult or costly to remove;
- Structural renovation work is typically constrained by the need for continuity of building operations;

- Generally, renovation of a steel-framed structure is more economical than the renovation of a concrete structure;
- The use of steelwork in a renovation is generally more cost-effective than the use of reinforced concrete; and
- The level of ductility of the existing construction may limit its strength.

These factors lead to fundamental differences in the approach to blast hardening between new and existing construction.

Uncertainty about existing construction may limit the sophistication of blast analysis that is appropriate; there may be no point in a precise determination of the presumed behavior where no equally precise understanding of the existing structure or its connections is available.

Conversely, the high cost of renovating an existing structure may justify a more sophisticated blast analysis where reliable detailed information is available and where there is reason to believe that substantial savings may be achieved in the construction cost of the strengthening project.

The approach that one takes with the analysis, or the design, is a matter of effectively relating the scale of the enormous, but transient, blast pressures to the effective resistance of the structure.

In considering the construction cost of retrofitting an existing facility, it is axiomatic to consider the total construction cost, not simply the structural costs. Often, the non-structural costs will equal or exceed the structural costs; therefore, the true costs of a retrofit project relate more to the number of locations of work than to the amount of work done in each location. This relationship should influence structural design and analysis decisions. For example, sophisticated analysis that reduces, but does not eliminate, the reinforcement of an inaccessible column may have little real benefit.

In new blast-resistant construction, ductile structural systems are designed to deform inelastically under large blast-induced forces. In many instances, existing construction will have limited post-elastic dynamic capability. Often, performance is limited by the shear capacity of critical structural elements. Further, wide-spread strengthening of the construction may be precluded by the costs of removing and replacing the enclosing construction.

In steel structures, common deficiencies include susceptibility to local buckling of outstanding flanges, and lack of connection ductility. Strengthening of a limited number of structural elements is usually practical, and, as with other types of renovations, there is a relative ease of working with steel construction.

In cast-in-place concrete structures, ductility and strength are often limited by the amount and the detailing of the existing reinforcement. For columns, fiber wraps, steel plate encasement and reinforced concrete encasement alternatives are practical. In most cases, wide-spread strengthening of a cast-in-place concrete structure will be impractical. Enhancement of a limited number of structural elements, however, is usually feasible.

Generally, precast concrete structures will exhibit the worst extreme event performance characteristics of either steel and concrete construction with potential weakness both at connections and in the detailing of the reinforcement. Consequently, the potential for effectively strengthening precast concrete structures is relatively limited.

There are fundamental differences between new construction and structural renovation and these differences are equally fundamental for blast hardening projects. The best designs for blast hardening of existing facilities are based on a clear vision of the overall security goals of the project and on an equally clear understanding of the detailed limitations of that which exists.

LESSONS LEARNED FROM THE 1993 BOMBING OF THE WORLD TRADE CENTER

At 12:18PM on Friday February 26th 1993, a 1000 lb. TNT equivalent bomb, located within a van parked immediately adjacent to the columns of the south wall of the north tower, was detonated in the second basement of The World Trade Center complex.

The resulting explosion destroyed approximately 30,000 sq. ft. of the concrete flat slab floor construction located at the first and second basements and badly damaged over 25,000 sq. ft. of construction. The structural steel framing above suffered far less damage. A small opening was breached within one bay at street level and the plaza level framing was left with a prominent upward bow.

This destruction occurred outside of the footprint of the towers. The tower structures were largely unscathed, with the north tower suffering minor damage to one column and the loss of two diagonals located immediately adjacent to the blast. The primary structure of the south tower suffered no damage. At no time was the structural integrity of either tower significantly impacted.

Conversely, several steel columns that supported the north end of the 22-story Vista Hotel were rendered un-braced for a 68 foot height by the loss of the concrete flat slabs in the basements. In the days immediately following the explosion, one of the most critical structural repair initiatives was to temporarily stabilize these columns. This was accomplished through the installation of tubular steel bracing.

From the perspective of performance of existing structural systems in an extreme event, i.e. systems not explicitly designed for blast resistance, two clear lessons can be learned from this tragic event:

1. The inherent blast resistance of an existing structure will be highly dependent on the scale of the building and on the scale of the wind or seismic forces for which the building has been designed; and
2. It is likely that steel construction will perform better than concrete construction because of its greater inherent ductility.

The structures of the 110-story World Trade Center towers were essentially unaffected by the blast, while the stability of the 22-story Vista Hotel was put into jeopardy. Fundamentally, this difference is a function of the relative scale of the buildings and the type of construction within the footprints of each building.

The inherent difference in blast-resistance between the steel and concrete construction was apparent to those of us who worked on the reconstruction. All around the blast site, one observed individual steel beams and columns remaining where the surrounding concrete slabs or beams were destroyed. Where steel beams failed, one observed in the dismembered pieces dramatic evidence of the ductile behavior that preceded failure.

Also, we observed large sections of concrete floor slab suspended in mid-air by one or two reinforcing bars, evidence of both the value of inherent tensile strength and the absence of this strength that is often the weakness of existing concrete construction not designed for extreme events.

GENERAL APPROACH TO HARDENING EXISTING STRUCTURES

Risk assessment and structural vulnerability assessment

While risk analysis and vulnerability assessment are essential first steps in any security project, these steps take on a special importance for an existing facility. The structural engineering vulnerability assessment of the existing facility needs establish the global strength of the lateral load resisting system relative to the magnitude of potential extreme events. Further, the particular vulnerabilities of specific structural systems and elements need be identified.

The importance of an overall vulnerability assessment of the structural systems of an existing facility needs to be communicated to the client at the beginning of a project. A certain prominent feature of the structure, such as a column, that the client wishes to protect may well be the reason an evaluation was sought; however, the loss of a related transfer truss or girder may precipitate a similar or larger collapse.

First, one needs begin with a condition assessment of the building. Ideally, this assessment should include review of the original construction drawings, shop drawings, subsequent renovation drawings, and maintenance records. Commonly, however, the available documentation is limited. Where relevant information is lacking, a prudent assessment report may qualify the pertinent conclusions. Following a review of the documentation, one or more visits to the facility, complemented by probes at critical locations, are usually appropriate. In some circumstances, a consolidated set of drawings is created. An additional component of the assessment of the building is a load survey. We sometimes conduct load surveys in existing buildings where we are transferring columns. When it comes time to study an element or a systems demand-to-capacity ratio the data from the survey can be very useful.

With the structural condition assessment of the existing building complete, a structural vulnerability assessment may be made for either an undefined threat or series of defined threats. Regardless, one should neither finalize the structural vulnerability assessment, nor proceed with a hardening design, until the risk assessment is completed.

Preliminary Analysis - Getting into scale

Typically the risk analysis addresses the possibility of threats from explosives ranging in size from a pipe bomb up to a fully laden tractor trailer. Depending upon the project's original design criteria, i.e. which natural hazards were considered, the height of the building, and the lateral load resisting system, it is possible that the global strength of a building's structure may be overwhelmed by the larger threat scenarios.

For buildings where the global structural system may be easily compromised, alternatives to structural hardening are the preferred course of action. Often, by relocating high-risk functions, the building threat may be reduced to manageable proportions. For buildings were the global systems are able to withstand the range of forces triggered by the threat with modest intrusions, we then turn our attention to an evaluation of redundancy and specific potential weaknesses in the structure.

For the structure assessment, the following needs to be considered:

- local behavior of the items that make up the element, such as the flange of a column;
- global behavior of an element; such as large deflection of a column unable to restrain the resulting P-delta forces;
- local behavior of the connections; such as net section limitations at bolt hole lines;
- local behavior of transfer systems and trusses; and
- global behavior of lateral load resisting systems.

To analyze the effects of the blast pressures on the varying elements one of the more practical resources is the Army's TM 5-1300 "Structures to Resist Accidential Explosions" (Army TM 5-1300, 1990). This reference, in combination with the ConWep Software (USAEWES/SS-R, 1992)[1] guide you in the calculation of the pressure and related information necessary to perform an analysis for reinforced concrete construction and for structural steel construction. There is additional software and reference guides available, FRANG (NCEL, 1988), FRANG (NCEL, 1989), and even expanded code finite element analysis software like Weidlingers' FLEX, but availability, cost and performance vary considerably. In some cases, a hand calculation is sufficient. At other times the use of advanced software is necessary. For those entering the field, a good reference is *Structural Dynamics* (Biggs, 1964).

[1] Dr. Theodore Krauthammer provided an overview of available software during the "Modern Protective Structures Short Course" sponsored by SEAoNY in June 2003.

Consideration of Alternatives – Both Structural and Non-Structural

Following the preliminary vulnerability analysis the design team meets to discuss the range of non-structural alternatives and the strengthening alternatives for the project. Often, the final criteria for the strengthening of the building are decided at this meeting.

Consideration must be given to the weight of the new structure that is being added to the building. A significant increase of weight to the building, such as will accompany the addition of concrete encasement or concrete walls, may require a supplementary reinforcement of the gravity or lateral load resisting systems of the structure. Care need be taken in checking with the local jurisdiction about the applicable building code to use in the case where you are reanalyzing the lateral load resisting system of the building. As with any renovation or retrofit project, the possibility of triggering a comprehensive upgrade of the existing building to the current building code needs to be considered. In almost all cases, accomplishing such an upgrade will not be economically or functionally feasible.

Detailed design and the impact of connections

As with most renovations, the cost impact on the project is directly related to the number of locations within the building requiring work. For most hardening, steel elements provide more cost effective results than concrete. The connection of choice for field installation of steel work is welding. For detailed design, we typically turn to steel for its many advantages. As with most renovations, steel provides compact, high strength, and ductile elements which in turn bring an ease of installation and attachment.

When reviewing existing structural elements, we typically find that the connections are the first limitation to the elements performance under extreme loads. Strengthening of connections at beam-to-column joints afford another leap in capacity for the system.

COMMON GOALS AND SITUATIONS

With rare exception, both owners and architects are seeking a more secure building that does not take on the ‘bunker’ aesthetic. These goals can be significant hurdles until after the risk assessment has effectively communicated limited hardening objectives based on limiting low probabilistic threats.

The hardening goals may include hardening of the room/area used for mail handling/inspection, hardening of the security barrier at entry and hardening of specific structural elements that are deemed vulnerable.

Protecting specific structural elements

The damage or loss of an individual member of a transfer system or truss may result in a disproportionate degree of damage to the building structure. Hardening of these elements takes the form of connection reinforcement, element reinforcement and stiffening of buckling-prone portions of the elements, e.g. individual plates of an unfilled built-up box column.

Where the design of the site layout or underground facilities provides for key elements, such as columns, to be accessible to attack, element hardening may depend upon the space available. Where possible the entire architectural envelope around the column may be utilized for increasing the strength of the element. In some locations, there also may be room within the depth of the ceiling to provide bracing to increase the buckling strength.

Providing redundancy to structural systems

For most existing buildings strengthening of elements is costly. It can often be highly intrusive and provide only localized protection. Providing load paths or system redundancy to the critical structural systems is another means of strengthening. In this case, one assumes that a critical element of a system is lost in the event and its loads need be shed to an adjacent system. For some buildings this may take the form of simply interconnecting a series of existing trusses, girders and columns such that they act together to effectively redistribute loads. For other buildings a completely new system may need to be inserted.

Hardening a specific room or area

Mail rooms are examples of rooms which can be subject to internal explosions. Strengthening for these areas includes the walls, floors, and ceilings. Venting systems are often an efficient method of mitigating the effects of blast in the hardened room and beyond. Strengthening in combination with a venting system is often your most efficient solution.

For some agencies, their building complex may include high risk functions that are directly adjacent to areas of public access. These include areas built below open air plazas, or buildings which are built with minimal setbacks from the public roads. External explosions outside of these areas are unable to be mitigated by increasing stand-off distance unless the local authority is willing to reroute traffic (highly unlikely). In these areas, the first response is to inquire about moving the high-risk agency or function away from the area. When that is not able to occur, one looks for blast curtains or sacrificial walls to shield the area from the direct pressures of a blast.

Hardening a secure perimeter or entry

Providing a secure perimeter takes the form of a combination of building steps, blast or sacrificial walls, and bollards. Glass in lobbies presents a challenge in trying to protect lobby occupants. Retrofitting with a puncture resistant interlayer and stronger glass lites usually is too costly. Reinforcement of just the glass can be short-sighted; where the grip of the mullions to the glass is too small, a single large projectile can be created in place of many smaller projectiles.

SUMMARY

A comprehensive security risk analysis that is clearly communicated to the client is an essential prerequisite to the blast hardening of an existing facility. Often, non-structural security measures combined with partial blast hardening are the appropriate response to the perceived threat.

The existing architectural and building services systems and the operational requirements of existing facilities place practical limitations on the hardening of existing facilities.

The relative scale of the extreme event compared with the wind and seismic criteria for which an existing facility was designed may limit the potential hardening objectives that can be achieved. Similarly, the degree of ductility inherent in the existing structural framework represents another possible limitation.

Common partial hardening objectives include the protection of critical rooms or areas, the hardening of a secure perimeter or entry and the hardening of specific structural elements or systems that are required to maintain overall stability of the structure.

The best designs for blast hardening of existing facilities are based on a clear vision of the overall security goals of the project and an equally clear understanding of the detailed limitations of that which exists.

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DESIGN OF STEEL STRUCTURES FOR BLAST-RELATED PROGRESSIVE COLLAPSE RESISTANCE

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Ronald Hamburger Andrew Whittaker

BIOGRAPHIES

Ron Hamburger: Ronald Hamburger is a structural engineer and principal with Simpson Gumpertz & Heger in San Francisco. Mr. Hamburger has nearly 30 years of experience in structural design, evaluation, upgrade, research, code and standards development, and education. Mr. Hamburger serves as chair of the BSSC Provisions Update Committee, AISC's Connection Prequalification Review Panel, and is vice-chair of the AWS D1.1 Seismic Task Committee. Further, he is a member of the ASCE-7 committee, is President-Elect of the National Council of Structural Engineering Associations and is the Project Director for the ATC-58 project on performance-based earthquake engineering. Mr. Hamburger was a member of the joint FEMA/ASCE Building Performance Assessment Team that studied the collapse of New York's World Trade Center on September 11, 2001.

Andrew Whittaker: Andrew Whittaker is an Associate Professor of Civil Engineering at the University at Buffalo, with research and design-professional interests in earthquake and blast engineering. He is a licensed Structural Engineering in the State of California. Dr. Whittaker serves as a member of the ASCE committees that address loadings on structures, blast engineering, and earthquake protective systems, ACI Committee 349 on reinforced concrete nuclear structures, BSSC Technical Subcommittee 12 on seismic isolation and passive energy dissipation systems for new buildings, as Vice President of the Consortium of Universities for Research in Earthquake Engineering, and as the Structural Team Lead for the ATC-58 project on performance-based earthquake engineering.

ABSTRACT

Structural steel framing is an excellent system for providing building structures the ability to arrest collapse in the event of extreme damage to one or more vertical load carrying elements. The most commonly employed strategy to provide progressive collapse resistance is to employ moment-resisting framing at each floor level so as to re-distribute loads away from failed elements to alternative load paths. Design criteria commonly employed for this purpose typically rely on the flexural action of the framing to redistribute loads and account for limited member ductility and overstrength using elastic analyses to approximate true inelastic behavior. More efficient design solutions can be obtained by relying on the development of catenary behavior in the framing elements. However, in order to reliably provide this behavior, steel framing connections must be capable of resisting large tensile demands simultaneously applied with large inelastic flexural deformations. Moment connections prequalified for use in seismic service are presumed capable of providing acceptable performance, however, research is needed to identify confirm that these connection technologies are capable of reliable service under these conditions. In addition, some refinement of current simplified analysis methods is needed.

INTRODUCTION

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

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

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

DESIGN STRATEGIES

Typical design strategies for collapse resistant buildings involve removal of one or more vertical load carrying elements and demonstrating that not more than specified portions of the building will be subject to collapse upon such occurrence. The element removal could occur as a result of any of several loading events including blast, vehicle impact, fire, or similar incidents. Regardless, the design strategy can be traced to lessons learned from observation of the blast-induced collapse of the Alfred P. Murrah Building in Oklahoma City. As illustrated in Figure 1 (Partin 1995) extreme damage to columns at the first story of the building, led to progressive collapse of much of the structure (Figure 2).

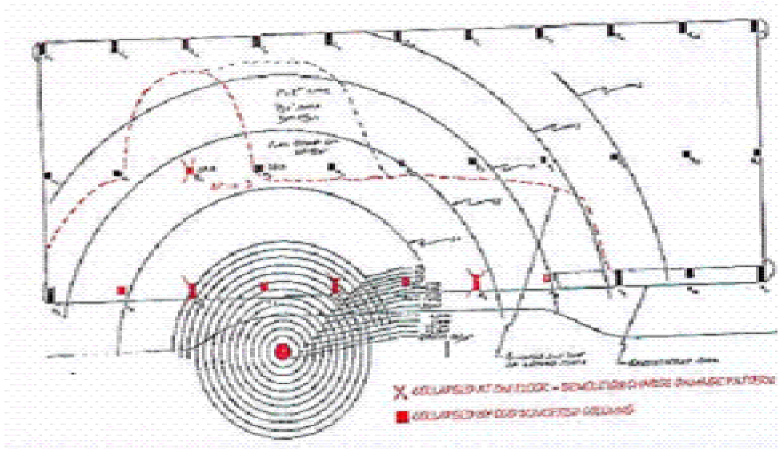

Figure 1 - Diagram showing elements damaged by initial blast adjacent to Murrah Federal Building

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

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

Moment-resisting steel frames are ideally suited to provision of this continuity and in avoiding progressive collapse. Three examples of the effectiveness of moment-resisting steel frames in arresting collapse and preventing progressive collapse as a result of extreme localized damage can be observed in the performance of buildings at New York's World Trade Center following the terrorist attacks of September 11, 2001. Figure 3 is a view of the north face of the North Tower of the World Trade Center, clearly indicating that the closely spaced columns and deep girders of the moment-resisting steel frame that formed the exterior wall of the structure was capable of bridging around the massive local damage caused by impact of the aircraft and arrest global collapse of the structure for nearly 2 hours. Figure 4 illustrates that the more conventional moment-resisting steel frame of the Deutsche Bank Building allowed that structure to arrest partial collapse induced by falling debris from the south tower of the

World Trade Center, despite the fact that an entire column was removed from the structure over a height of 10+ stories. Figure 5 is a plan view of the WTC-6 building at New York's World Trade Center following collapse of the North Wall of the North Tower across the top of the building. A series of one-bay moment-resisting steel frames placed around the perimeter of the building arrested collapse and limited collapse to areas not protected by moment-resisting framing.

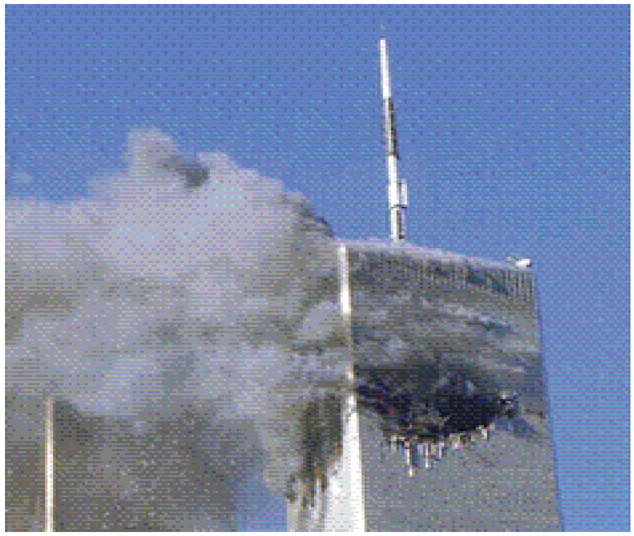

Figure - 3. North Tower of World Trade Center, Illustrating the ability of the perimeter frame to bridge around the massive aircraft impact damage and arrest progressive collapse.

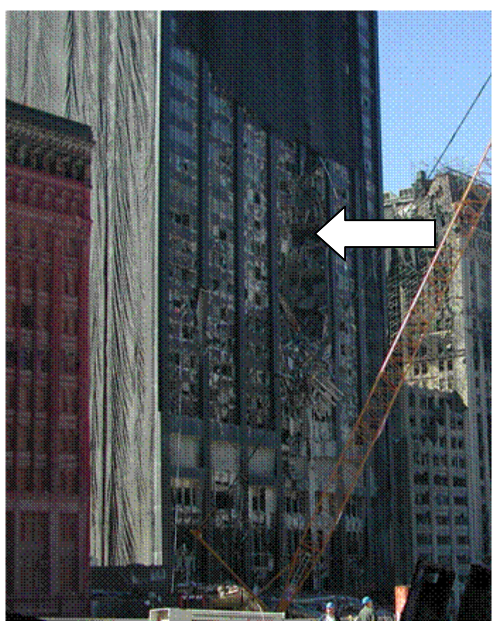

Figure - 4. Deutsche Bank Building remains standing despite column loss over multiple stories (see arrow)

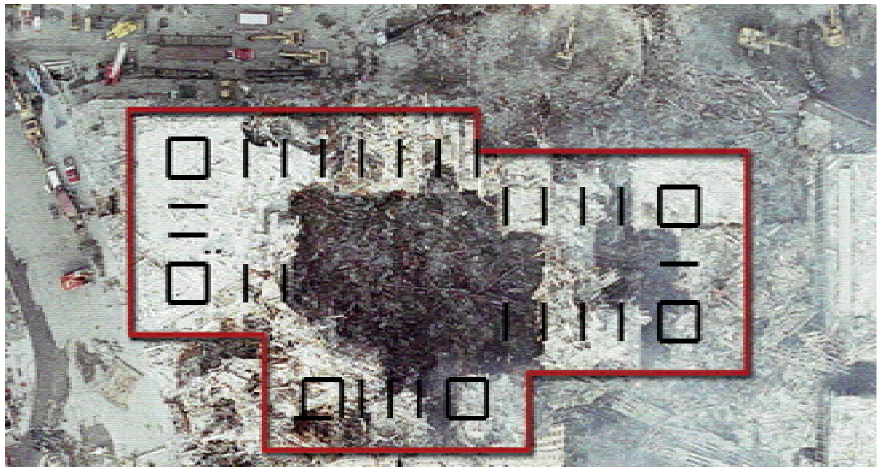

Figure 5 - Collapse of World Trade Center 6, induced by falling debris from the North Tower. Note that the dark lines indicate approximate locations of one-bay steel moment frames around building perimeter.

The use of moment-resisting steel framing to provide collapse resistance is an obvious choice. Figure 6 illustrates how a building with a continuous moment-resisting steel frame on each line of columns can resist collapse through redistribution of load to adjacent columns. Simplified guidelines for the design of such systems have been developed for the U.S. General Services Administration (ARA, 2003) and are available to designers engaged in the design or review of federal facilities. These guidelines specify that elements of the frame be proportioned with sufficient strength to resist twice the dead load and live load anticipated to be present, without exceeding inelastic demand ratios obtained from the federal guidelines for seismic rehabilitation of buildings (ASCE, 2002). The design model utilized in these simple procedures is conceptually incorrect, but probably provides adequate design solutions.

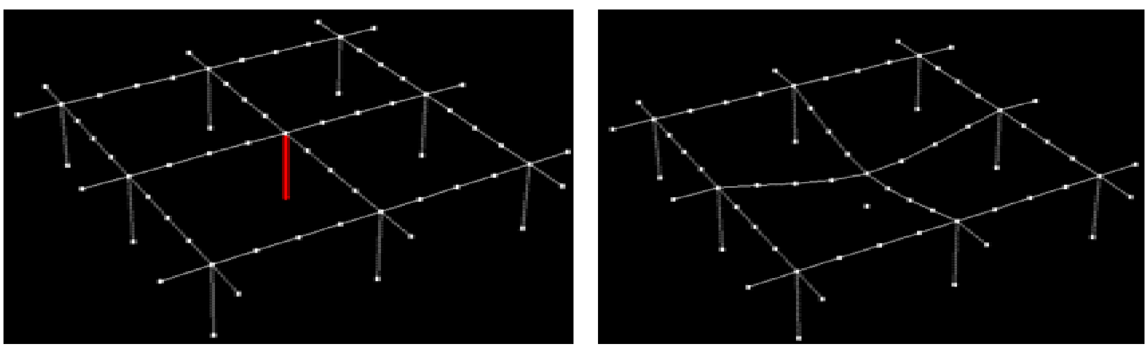

Figure 6 - Redistribution of gravity loads from removed column in building with a continuous moment-resisting steel frame along column lines.

Under this design model, the beams and columns are assumed to be required to distribute twice the vertical forces initially resisted by the removed element, through flexural behavior. The elements are required to be proportioned to resist twice the load initially resisted by the "removed" element based on theory related to the instantaneous application of load on an elastic element. Figure 7 and Figure 8 are respectively, displacement vs. time and force vs. time plots from a response history analysis of an elastic single degree of freedom structure. The structure has a natural period of vibration of 0.5 seconds, a stiffness of 100 kips/inch and moderate damping. A load of 100 kips is instantaneously applied to the structure. In response to this the structure experiences an instantaneous deflection of 2 inches, then oscillates with slowly decaying amplitude until a steady state deflection of 1 inch is approached. The maximum force in the structure is 200 kips, or twice the statically applied amount and the maximum deflection of the structure is 2 inches, or twice the static value, resulting in the impact coefficient of 2 used in the federal progressive collapse design guidelines.

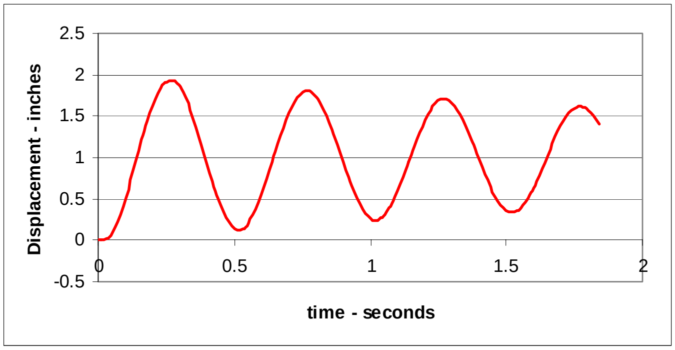

Figure 7 – Elastic Displacement Response of a Structure with Instantaneous Load Application

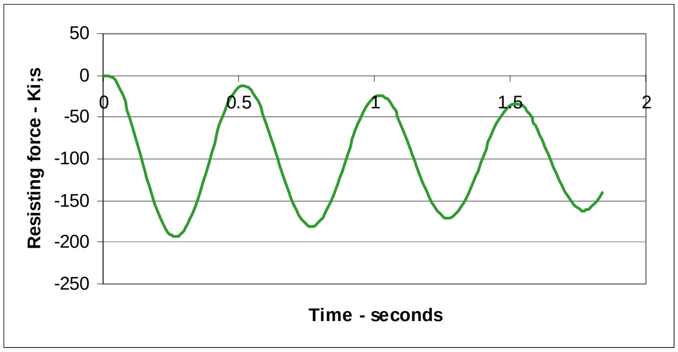

Figure 8 – Elastic Strength Demand on Structure with Instantaneous Load Application

Under the federal progressive collapse design guidelines, members are permitted to experience "flexural inelasticity" based on permissible values contained in seismic guidelines recognizing that the amplified loading occurs for a very short duration and that long term loading following removal is a static condition. Specifically, compact framing is considered acceptable if the ratio of moment computed from an elastic analysis (M_A) to the expected plastic moment capacity of the section (M_{pE}), is less than 3. Noncompact sections are permitted to be designed with a limiting ratio M_A/M_{PE} of 2. Figure 9 is a plot of displacement vs. time from a nonlinear response history analysis of the same structure analyzed previously except that it has been assumed that the structure has a limiting plastic strength of 120 kips. Using the procedures in the federal guidelines, the M_A/M_{PE} ratio for this structure would be (*2* x *100* kips / *120* kips) or 1.66, which would be within the permissible level of inelasticity either for compact or noncompact sections. As can be seen, the ratio of maximum displacement to steady state displacement increases to more than 2.5 for this structure.

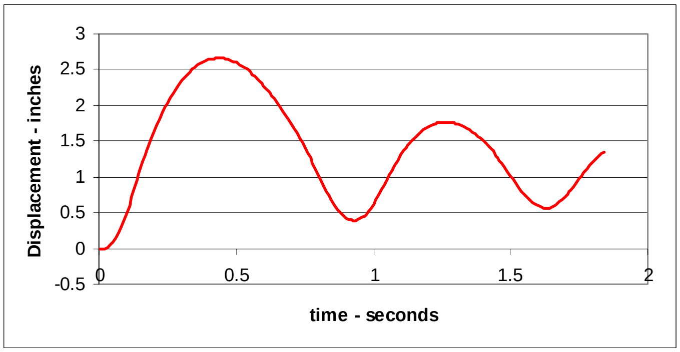

Figure 9 – Inelastic Displacement Response of a Moderately Strong Structure with Instantaneous Load Application

Figure 10 is a plot from a similar analysis, in which the strength of the structure has been further decreased to 80 kips. In this case, the ratio of M_A/M_{PE} is 2.5, more than permitted for noncompact sections but less than the value of 3, permitted for compact sections. As can be seen, the structure is subject to unlimited increasing displacements, or stated simply, collapses. This identifies a basic flaw in the federal progressive collapse guidelines. While it should clearly be permissible to permit some inelastic deformation of framing used to resist collapse, as measured by the M_A/M_{PE} ratio, the structure must, as a minimum, have sufficient plastic strength to support the weight of the structure, in a static condition. The structure illustrated in Figure 10 did not have this strength. The federal progressive collapse guidelines do not actually require this evaluation but should.

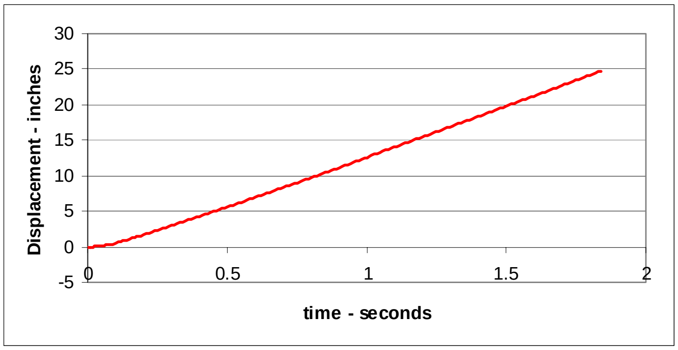

Figure 10 – Inelastic Displacement Response of a WeakStructure with Instantaneous Load Application

Fortunately, the assumption that load redistribution occurs through flexural behavior alone is very conservative and results in the design of members that are much larger than actually required to resist progressive collapse. Figure 11 illustrates an alternative load resisting mechanism for redistribution of load that relies on catenary behavior of the steel framing and compressive arching of the concrete floor slab. In the top illustration in this figure, the frame is supporting loads prior to column removal. In the middle illustration the central column has been removed beneath the floor and the frame is redistributing loads to the outer columns through flexure, as the floor locally falls downward. If the girders are not sufficiently strong to resist the strength demands resulting from the instantaneous removal of the central support column in an elastic manner, which is what is inherently assumed by the federal guidelines, plastic hinges will from at the two ends of the beams and in the mid-span region, near the removed column. Neglecting loading along the beam span, the two-span beam will have a strength equivalent to $8M_p/L$, where M_p is the plastic moment capacity of the beam and L is the distance between the outer columns, to resist the load imposed on the beam by the now discontinuous central column and to slow the downward movement of the floor system.. If this strength is not sufficient to accomplish this, the beam will deflect sufficiently to mobilize catenary tensile action, which if sufficient, will eventually arrest the collapse. This mode of behavior, which is not explicitly considered in the federal guidelines, but is relied upon, is illustrated in the lower figure where the beam has formed plastic hinges at the beam-column joints and is now resisting loads from the interior column through catenary tensile behavior of the beam, balanced at the columns by compressive action in the slab. In fact, if the beam were compact, and laterally supported, the federal guidelines would permit the beam to arrest the collapse of a central column load with a magnitude as high as $12M_p/L$. Clearly, in such a case, even though neglected by the federal guidelines, either catenary tensile behavior will be mobilized or the structure will fail to arrest collapse..

Most designs presently neglect, at least explicitly, the ability to develop catenary behavior and implicitly rely solely on the flexural mechanism. As an illustration of the potential efficiency of the catenary mechanism, in a recent study, it was determined that in a structure with 30 foot bay spacing, ASTM A992, W36 horizontal framing could safely support the weight of nearly 20 stories of structure above in the event of column removal (Hamburger, 2003), although deflection would be significant. There are several potential implications of this finding. First, it is not necessary to provide moment resisting framing at each level of a structure, in order to provide progressive collapse resistance. Second, it is not necessary to have substantial flexural capacity in the horizontal framing, either in the beam section itself or in the connection, in order to provide this collapse resistance. Third, it may not be necessary to provide full moment resistance in the horizontal framing and conventional steel framing may be able to provide progressive collapse resistance as long as connections with sufficient tensile capacity to develop catenary behavior are provided.

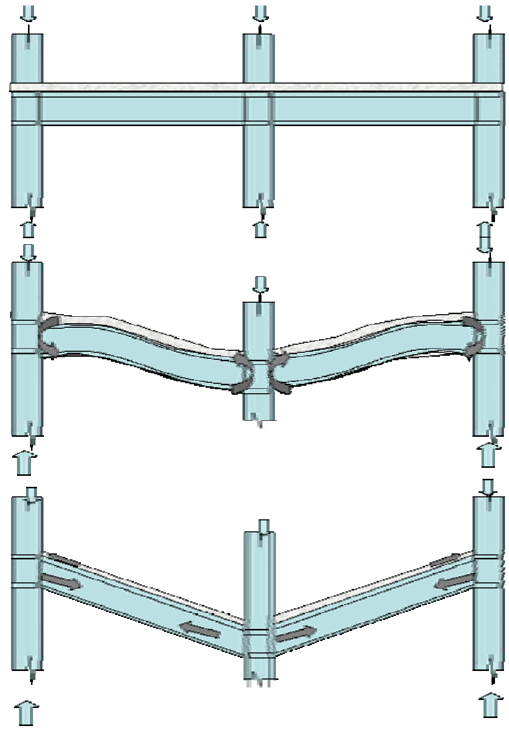

Figure 11 - Load-resisting mechanisms upon removal of a supporting column

DESIGN APPLICATIONS

As an example of the efficiency of moment-resisting steel frame structures in the resistance of progressive collapse a study was conducted of the cost premium associated with providing progressive collapse resistance in a typical structure. In this study, a structure with a regular 30-foot grid pattern was reviewed. The floor system was comprised of 3-inch, 20-gauge metal deck, supporting a 5-1/2 inch (total thickness) lightweight concrete slab, with non-composite floor beams. Initially framing was designed without moment-resistance. The resulting framing, as illustrated in Figure 12 used W18x40, A992 beams and W24x62 A992 girders. Next, the beams and girders along column lines were assumed to be provided with moment-resistance and an evaluation of the structure for ability to resist instantaneous removal of a single interior column was performed using the federal progressive collapse guidelines. It was determined that the maximum value of M_A/M_{PE} in the framing was 1.5, or half the permissible value for compact sections. Thus, it was determined that progressive collapse resistance can be achieved in steel moment frame structures without increase in the weight of the framing.

Although collapse resistance can be provided without weight increase, there is, of course, a significant cost premium associated with the provision of moment connections between every beam, girder and column. therefore, an additional study was performed to determine if the number of moment-resisting connections in the building could be reduced. As a first step in this process, it was determined that if the moment-resistance was not provided for the W18x40 beams on the column lines but was provided for the W24x62 girders, the maximum value of M_A/M_{PE} is increased only to 1.9, which is still well within the limits permitted by the guidelines. Next, it an investigation was preformed to determine if it would be possible to provide the desired resistance to collapse by providing moment resistance on only a few of the floors in a multi-story buildings. It was determined that by using W36x300 sections as the beams and girders at one floor level, it would be possible to provide progressive collapse protection for as many as 15 supported stories. This results in very few moment connections and a total increase in framing weight of about 1.5 pounds per square foot, demonstrating that very economical solutions for providing collapse resistance in steel structures is possible.

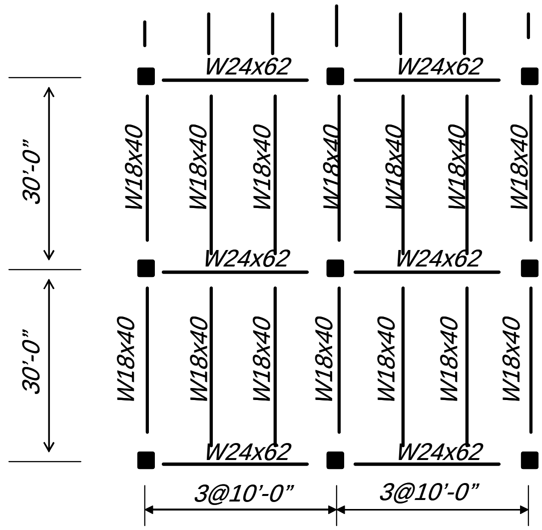

Figure 12 – Typical Floor Framing Evaluated for Collapse Resistance

RESEARCH NEEDS

While the use of catenary behavior to provide progressive collapse resistance holds great promise for steel structure design, it is not immediately apparent what types of connections of beams to columns will possess sufficient robustness to permit the necessary development of plastic rotations at beam ends together with large tensile forces. Figures 13 and 14 are pictures of bolted web–welded flange moment resisting connections that fractured in the 1994 Northridge earthquake. These fractures occurred at beam column joints at an estimated drift demand of approximately 0.01 radian.

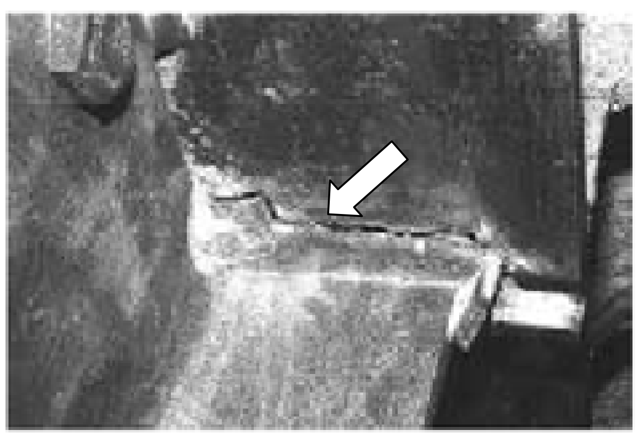

Figure 13 - Fractured welded beam flange to column flange connection in building discovered following the 1994 Northridge earthquake

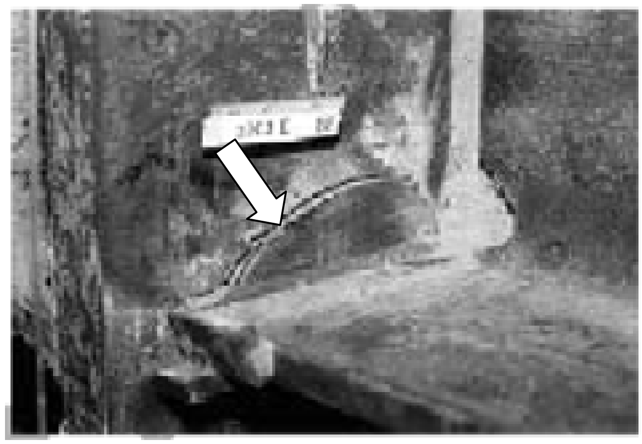

Figure 14 - Fractured welded beam flange to column flange joint discovered following the 1994 Northridge earthquake.

Mobilization of catenary action in framing may require plastic rotations on the order of 0.07 radians or more. It is true that there are substantial differences in the loading demand that occurs on beam-column joints in an earthquake as compared to those that occur in a frame resisting progressive collapse. Earthquake demands are cyclic and induce low-cycle fatigue failure of connections. However, demands applied on members and connections when resisting direct air-blast loadings might produce somewhat high strain rates, may be of larger magnitude and will occur simultaneously with large axial tension demands. Under conditions of high strain rate, steel framing becomes both stronger but more brittle. There is evidence that standard beam-column connection framing is quite vulnerable to such loading. Figure 15 is a photograph of a failed beam-column connection in the Deutsche Bank building. The beam which connected to the column using a bolted flange plate type connection was sheared directly off the column due to the impact of debris falling onto the structure from the adjacent collapsing South Tower of the World Trade Center. Also visible at the bottom of this picture is failure of the bolted column splice. Figure 16 is a picture of a failed bolted shear connection in the World Trade Center 5 building that resulted from development of large tensile forces in the beam due to fire effects. Clearly, these failures indicate that standard connection types commonly used in steel framing today may not be capable of allowing the structure to develop the large inelastic rotations and tensile strains necessary to resist progressive collapse through large deformation behavior. Despite these poor behaviors, it is also known that when properly configured and constructed, using materials with appropriate toughness, steel connections can provide outstanding ductility and toughness. Figure 17 illustrates the deformation capacity of beam-column connections designed with appropriate configurations and materials.

Following the damage experienced in steel buildings in the 1994 Northridge earthquake an extensive program of investigation was undertaken to develop beam-column connections capable of providing reliable behavior under the severe inelastic demands produced by earthquake loading. A number of connection configurations capable of acceptable behavior were developed (SAC 2000a). In parallel with these connection configurations, a series of materials, fabrication and construction quality specifications were also produced (SAC 2000b). While these technologies have been demonstrated capable of providing acceptable seismic performance, it is unclear whether these technologies are appropriate to providing protection against progressive collapse. Indeed, some of the connection configurations presented in the SAC documents rely on relief of high stress and strain conditions in the beam-column connection through intentional reduction in cross section that could lead to other failures under high impact load conditions. However, it is also possible that less robust connections than those demonstrated as necessary for seismic resistance could be adequate to arrest collapse in some structures. The moment-resisting connections in the World Trade Center 6 building, for example, which were not particularly robust by seismic standards, were able to successfully arrest collapse of that structure.

Figure 15 - Failed beam-column connection in Deutsche Bank Building

Figure 16 - Failed bolted high strength shear connection in World Trade Center 5

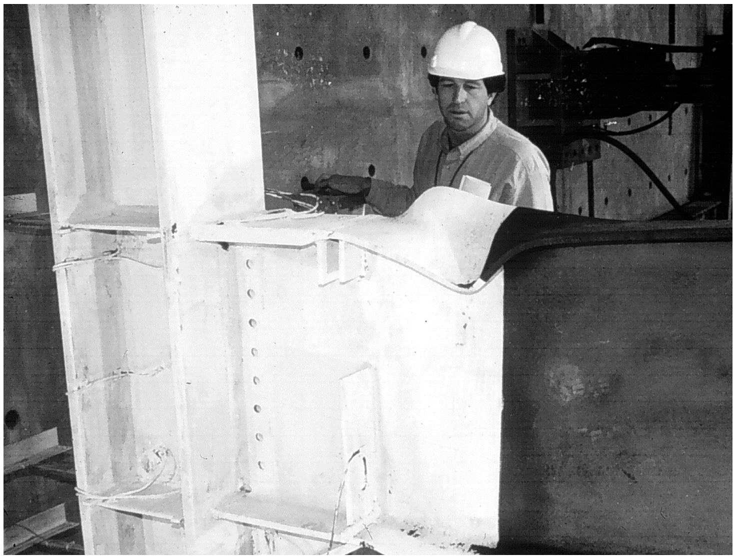

Figure 17 -. Extreme plastic deformation of beam-column connection designed for enhanced inelastic behavior

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

ACKNOWLEDGEMENT

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PERFORMANCE-BASED ENGINEERING OF BUILDINGS FOR EXTREME EVENTS

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BIOGRAPHIES

Andrew Whittaker: Andrew Whittaker is an Associate Professor of Civil Engineering at the University at Buffalo, with research and design-professional interests in earthquake and blast engineering. He is a licensed Structural Engineering in the State of California. Dr. Whittaker serves as a member of the ASCE committees that address loadings on structures, blast engineering, and earthquake protective systems, ACI Committee 349 on reinforced concrete nuclear structures, BSSC Technical Subcommittee 12 on seismic isolation and passive energy dissipation systems for new buildings, as Vice President of the Consortium of Universities for Research in Earthquake Engineering, and as the Structural Team Lead for the ATC-58 project on performance-based earthquake engineering. Dr. Whittaker was a member of the 3-person MCEER-NSF reconnaissance team that collected forensic data at Ground Zero in the weeks after the September 11, 2001, terrorist attacks.

Ron Hamburger: Ronald Hamburger is a structural engineer and principal with Simpson Gumpertz & Heger in San Francisco. Mr. Hamburger has nearly 30 years of experience in structural design, evaluation, upgrade, research, code and standards development, and education. Mr. Hamburger serves as chair of the BSSC Provisions Update Committee and the AISC Connection Prequalification Review Panel, and is vice-chair of the AWS D1.1 Seismic Task Committee. Further, he is a member of the ASCE-7 committee, is President-Elect of the National Council of Structural Engineering Associations and is the Project Director for the ATC-58 project on performance-based earthquake engineering. Mr. Hamburger was a member of the joint FEMA/ASCE Building Performance Assessment Team that studied the collapse of New York’s World Trade Center on September 11, 2001.

Michael Mahoney: Michael Mahoney is a Senior Geophysicist with the Mitigation Division of the Department of Homeland Security’s Federal Emergency Management Agency. He currently serves as the Acting Director of the National Earthquake Program Office in Federal Emergency Management Agency. Since 1991, Mr. Mahoney has been responsible for many of the technical activities under the National Earthquake Hazards Reduction Program (NEHRP), including the FEMA/SAC Steel Moment Frame Buildings Study and FEMA’s current Performance Based Seismic Design initiative. From 1984 to 1991, Mr. Mahoney was with FEMA's Office of Loss Reduction, within the National Flood Insurance Program. From 1978 to 1984, he was employed as a Loss Prevention Consultant with Factory Mutual Engineering. He holds Bachelors and Masters Degrees in physics.

ABSTRACT

Blast, earthquakes, fire and hurricanes are extreme events for building construction and warrant innovative structural engineering solutions. The state-of-the-practice and new developments in performance-based earthquake engineering (PBEE) are discussed, with emphasis on hazard intensity measures, engineering demand parameters, and performance levels. The new performance-based earthquake engineering methodology is extended to performance-based blast engineering. Sample intensity measures, engineering demand parameters, and performance levels are proposed for blast engineering. Some similarities and differences between performance approaches for blast and earthquake engineering are identified.

INTRODUCTION

Performance-based engineering of buildings and infrastructure for gravity and windstorm loadings has been indirectly undertaken for more than 20 years since the introduction of strength (concrete structures) and load-and-resistance-factor (steel structures) design in the 1960s and 1970s. Such engineering of buildings and infrastructure has been based on force-based analysis and design checking of *components* using

$$\sum \alpha_i L_i \le \phi C \tag{1}$$

where α_i are load factors, L_i are load effects (e.g., dead load, live load), ϕ is a capacity reduction factor for the action that is being checked (e.g., moment, axial load, shear force) and C is the component capacity that is determined using a materials standard such as the AISC Load and Resistance Factor Design Manual (AISC 2002). Factored component *force* demands are required to be less than or equal to de-rated component *force* capacities. The values of α_i and ϕ were selected to ensure that the probability of component failure, measured here as component demands greater than component capacities, is extremely low. *Global* performance of a framing system is measured by performance at the *component* level. No statements are made regarding the relationship between *component* and *system* failure.

Extreme loadings on buildings and infrastructure are produced by natural and man-made hazards including strong earthquakes, hurricane and tornado winds, blast, fire and equipment malfunctions. Setting aside equipment malfunctions for the purpose of this paper, the extreme loadings of Figure 1 should be resisted by buildings and infrastructure without collapse for sufficient time so as to allow the occupants the time required to exit the structure.

a. Earthquakes

b. Blast

c. Fire

d. Hurricanes

Figure 1. Extreme loadings and effects on building structures

Blast and earthquake loadings are short-term loadings with durations measured in milli-seconds and seconds, respectively. For such loadings, component and system ductility can be utilized to avoid system collapse. For relatively long-duration loads such as hurricane wind loadings on buildings, strength alone must be used to avoid collapse. Because component and system ductility are related to framing system displacements and deformations, and not component forces, performance-based engineering for extreme blast and earthquake loadings must be displacement or deformation-based based rather than force-based per (1).

The following sections of this paper provide summary information on the state-of-the-practice in performance-based earthquake engineering and the framework for on-going and future developments in performance-based earthquake engineering. Aspects of the performance-based earthquake engineering framework that might prove useful in the development of performance-based guidelines for blast engineering of buildings are identified. Some similarities and differences between performance approaches for blast and earthquake engineering are identified.

PERFORMANCE-BASED EARTHQUAKE ENGINEERING OF BUILDING STRUCTURES

Practice of performance-oriented earthquake engineering

The traditional prescriptive provisions for seismic design contained in U.S. building codes (e.g., ICBO 1997; FEMA 2000b) and under development since the late 1920s (ATC 1995) could be viewed as performance-oriented in that they were developed with the *intent* of achieving specific performance, that is, avoidance of collapse and protection of life safety. It was assumed by those engineers preparing the codes that buildings designed to these prescriptive provisions would (1) not collapse in very rare earthquake; (2) provide life safety for rare earthquakes; (3) suffer only limited repairable damage in moderate shaking; and (4) be undamaged in more frequent, minor earthquakes. The shortcomings of the prescriptive procedures include fuzzy definitions of performance and hazard and the fact that the procedures do not include an actual evaluation of the performance capability of a design to achieve any of these performance objectives. Further, records of earthquake damage to buildings over the past 70+ years following minor, moderate and intense earthquake shaking has demonstrated that none of the four performance objectives has been realized reliably. Deficiencies in the prescriptive provisions in terms of accomplishing the four target objectives have been identified following each significant earthquake in the United States and substantial revisions to the prescriptive provisions have then been made.

Performance expectations for mission-critical buildings began to evolve in the mid-1970s following severe damage to a number of emergency response facilities, most notably hospitals, in the 1971 San Fernando earthquake. Earthquake engineers decided that those buildings deemed to be essential for post-earthquake response and recovery (e.g., hospitals, fire stations, communications centers and similar facilities) should be designed to remain operational following severe earthquakes, and assumed that this would be achieved by boosting the required strength of such buildings by 50% compared with comparable non-essential buildings and requiring more rigorous quality assurance measures for the construction of essential facilities[1]. Since that time, the prescriptive provisions have evolved slowly but still include few direct procedures for predicting the performance of a particular building design, or for adjusting the design to affect the likely performance, other than through application of arbitrary importance factors that adjust the required strength.

Large economic losses and loss of function in mission-critical facilities following the 1989 Loma Prieta and 1994 Northridge earthquakes spurred the development of performance-based seismic design procedures with the goal of developing resilient, loss-resistant communities. In the early 1990s, experts design professionals and members of the academic community, ostensibly structural and geotechnical engineers, recognized that new and fundamentally different design approaches were needed because the prescriptive force-based procedures were a complex compendium of convoluted and sometimes contradictory requirements, were not directly tied to the performance they were intended to achieve, were not reliable in achieving the desired protection for society, were sometimes excessively costly to implement, and were not being targeted at appropriate performance goals in most cases.

[1] Although the 50% increase in strength served to reduce damage to the structural framing, there is no evidence to support the assumption that the essential facility would be operational after severe earthquake shaking.

Funding in the early to mid-1990s from the Federal Emergency Management Agency (FEMA) to the Applied Technology Council (ATC) and the Building Seismic Safety Council (BSSC) led to the development of the *NEHRP Guidelines and Commentary for Seismic Rehabilitation of Buildings* (FEMA 1997). This development effort marked a major milestone in the evolution of performance-based seismic design procedures and articulated several important earthquake-related concepts essential to a performance-based procedure. The key concept was that of a performance objective, consisting of the specification of the design event (earthquake hazard), which the building is to be designed to resist, and a permissible level of damage (performance level) given that the design event is experienced. Another important feature of the *NEHRP Guidelines* (FEMA 273/274) was the introduction of standard performance levels, which quantified levels of structural and nonstructural damage, based on values of standard structural response parameters. The *NEHRP Guidelines* also specified a total of four linear and nonlinear analysis procedures, each of which could be used to estimate the values of predictive response parameters for a given level of shaking, and which could then be used to evaluate the building's predicted performance relative to the target performance levels contained in the performance objective. Figure 2 below illustrates the qualitative *performance levels* of FEMA 273/274 superimposed on a global force-displacement relationship for a sample building. The corresponding levels of damage are sketched in the figure. Brief descriptions of the building damage and business interruption (downtime) for the three performance levels are given in Table 1.

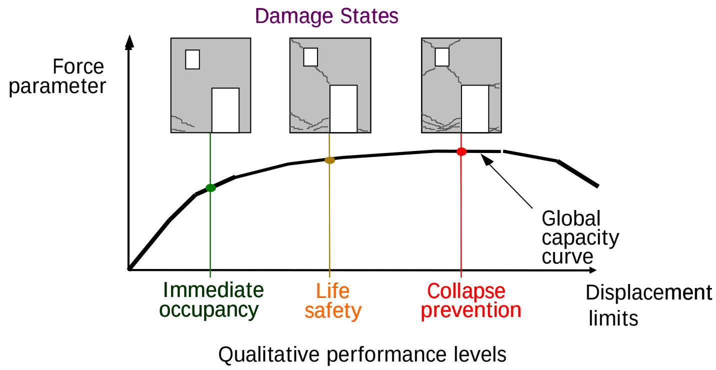


Figure 2. Qualitative performance levels of FEMA 273/274/356 (*after Comartin*)

Performance level	*Damage description*	*Downtime*
Immediate occupancy	Negligible structural damage; essential systems operational; minor overall damage	24 hours
Life safety	Probable structural damage; no collapse; minimal falling hazards; adequate emergency egress	Possible total loss
Collapse prevention	Severe structural damage; incipient collapse; probable falling hazards; possible restricted access	Probable total loss

Table 1. Building performance levels per FEMA 273/274/356 (*after Comartin*)

Figure 3 illustrates the FEMA 273/274 nonlinear static procedure for performance assessment. First, the earthquake hazard is characterized by one or more elastic acceleration response spectra. A nonlinear mathematical model of the building is prepared and subjected to monotonically increasing forces or displacements to create the capacity curve of Figure 3, which is generally plotted in terms of base shear (ordinate) versus roof displacement (abscissa). A maximum roof displacement is calculated for each design spectrum using an equivalent SDOF nonlinear representation of the building frame. Component deformation and force actions for performance assessment are then established for the given roof displacement using the results of the nonlinear static analysis. Component deformation and force demands are then checked against component deformation and force capacities, which are summarized for the performance levels of Figure 2 in the materials chapters of FEMA 273. If *component* demands do not exceed *component* capacities, the *building* performance objective are assumed to have been met.

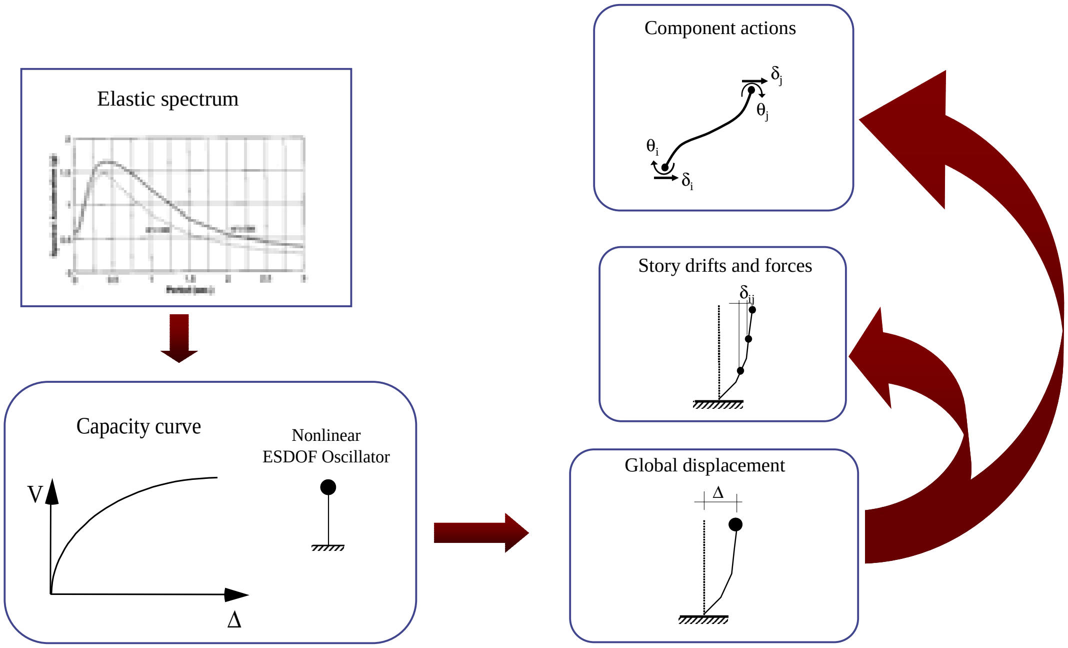


Figure 3. Performance assessment procedure of FEMA 273/274/356 (*after Comartin*)

Other projects including *ATC-40, Methodology for Evaluation and Upgrade of Concrete Buildings* and *Vision-2000 Framework for Performance-based Seismic Design Project* further developed and extended the technology developed in FEMA-273/274. These technologies were further refined by the American Society of Civil Engineers in their conversion of the FEMA-273/274 reports into the *Prestandard for Seismic Rehabilitation of Buildings, FEMA-356* (FEMA 2000a). Together, the *FEMA-356*, *ATC-40*, and *Vision-2000* documents define the current state of practice of performance-based seismic engineering.

Hamburger (2003) identified key shortcomings with the state of practice characterized by FEMA 273/356, including (1) the current procedures predict structural response and demands based on the global behavior of the structure but evaluate performance on the basis of damage sustained by individual components with the result that the poorest performing elements tend to control the prediction of structural performance, (2) much of the acceptance criteria contained in the documents, and used by engineers to evaluate the acceptability of a structure's performance is based on judgment, rather than laboratory data or other direct substantiating evidence, leading to questions regarding the reliability of the procedures, (3) many structural engineers view the guidelines as excessively conservative, when compared against designs developed using prescriptive criteria, however, the reliability of the guidelines and their ability to actually achieve the desired performance has never been established, and (4) the performance levels of FEMA 273/356 do not directly address some primary stakeholder concerns, that is probable repair costs and time of occupancy loss in the building, due to earthquake induced damage.

Following the discovery of unanticipated brittle fracture damage to welded moment-resisting steel frame buildings following the 1994 Northridge earthquake, FEMA sponsored a large project (widely known as the SAC Steel Project) to develop seismic evaluation and design criteria for that class of buildings. Key products of the project included a series of recommended design criteria documents [FEMA-350 (FEMA 2000c), FEMA-351 and FEMA-352], which incorporated performance-based design methodologies that addressed some of the issues associated with the state of practice per FEMA 273/356. The FEMA technical reports provided a large database of research data on the structural performance of this one structural system, which permitted the development of less subjective acceptance criteria for use in design, developed a methodology for evaluating the structural performance of a building based on its global response and behavior rather than solely on the amount of damage sustained by individual structural components, and developed a methodology for characterizing a level of confidence associated with a design's ability to meet a performance objective, addressing in part, concerns related to designer warranties of building performance (Hamburger 2003). Although the SAC performance methodology has not seen widespread acceptance, the prescriptive procedures contained in *FEMA-350* and *FEMA-351* that were validated using the performance-based methodology have been widely accepted and incorporated into national design standards and building codes.

Towards performance-based earthquake engineering

FEMA has contracted with the Applied Technology Council (ATC) to develop a *next generation* of performance-based seismic design guidelines for buildings, a project known as ATC-58. The guidelines are to be applicable to new and retrofit building construction and will address structural and non-structural components. Although focused primarily on design to resist earthquake effects, the next generation performance guidelines will be compatible with performance-based procedures being developed at this time for other hazards including fire and blast.

The ATC-58 project will utilize performance objectives that are both predictable (for design professionals) and meaningful and useful for decision makers. Preliminary project work tasks have revealed that these decision makers (or stakeholders) are a disparate group, representing many constituencies and levels of sophistication (Hamburger 2003). Decision makers include building developers, corporate facilities managers, corporate risk managers, institutional managers, lenders, insurers, public agencies and regulators. Each type of decision maker views performance from a different perspective and select performance goals using different decision making processes. The performance-based design methodology will include procedures for estimating risk on a design-specific basis, where risk will be expressed on either a deterministic (scenario basis or event) or a probabilistic basis. Risk will be expressed in terms of specific losses (e.g., cost of restoration of a facility to service once it is damaged, deaths and downtime) rather than through the use of traditional metrics (e.g., life safety in a design-basis earthquake).

The performance prediction process is similar to that utilized in the HAZUS national loss estimation software, although the individual steps in the process will be implemented differently. Figure 4 from Hamburger (2003) is the flow chart for the ATC-58 performance prediction methodology. Much of the methodology is based on procedures currently under development by the Pacific Earthquake Engineering Research (PEER) Center (Moehle 2003) with funding from the U.S. National Science Foundation.

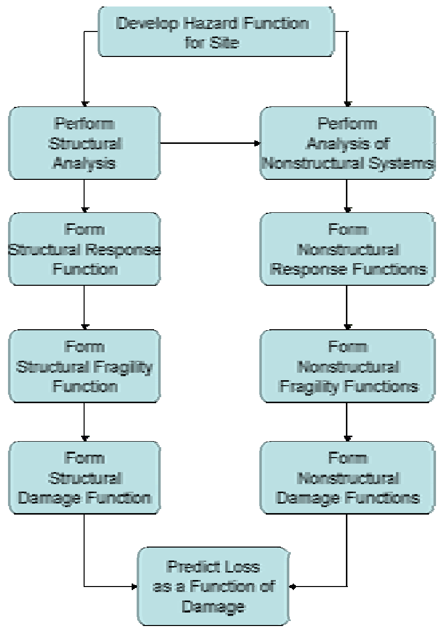


Figure 4. ATC-58 performance prediction flowchart (Hamburger 2003)

The PEER performance-based methodology is formalized on a probabilistic basis and is composed of four sequential steps: hazard assessment, structural/nonstructural component analysis, damage evaluation, and loss analysis or risk assessment. The product from each of these four steps is characterized by a generalized variable: Intensity Measure (*IM*), Engineering Demand Parameter (*EDP*), Damage Measure (*DM*), and Decision Variable (*DV*), for each of the steps, respectively. Figure 5 illustrates the methodology and its probabilistic underpinnings. The variables are expressed in terms of conditional probabilities of exceedance (e.g., $p(EDP \mid IM)$) and the approach of Figure 5 assumes that the conditional probabilities between the parameters are independent. Moehle (2003) and Hamburger (2003) describes the performance-based methodology that has been adopted for the ATC-58 project. Key features of the methodology as presented by Moehle and Hamburger are summarized below for a building of a given geometry and design (termed *D* in the figure) and location (termed *O* in the figure). As such, the building and the hazard are fully defined.

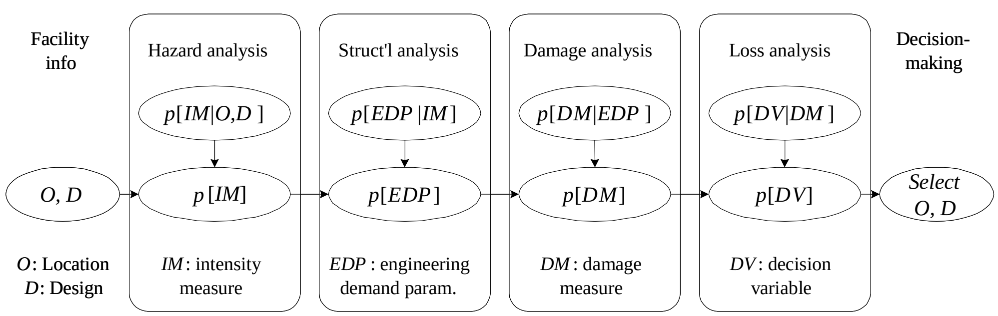


Figure 5. Probabilistic underpinnings of the PEER and ATC-58 performance methodologies (Moehle 2003)

Hazard analysis is the first of the four steps and produces ground motion Intensity Measures (*IM*s). The traditional IMs are peak ground acceleration and spectral acceleration at selected periods. Values for the *IM*s are obtained by probabilistic seismic hazard assessment at the location of the site (*O*). *IM*s are typically described as a mean annual probability of exceedance of the IM ($p\langle IM \rangle$ in Figure 5). The second step in the process is to use the *IM*s and the corresponding earthquake histories for simulation of the building response and the estimation of Engineering

Demand Parameters (*EDP*s). *EDP*s, which traditionally have included component forces and deformations and story drifts, are calculated by linear or nonlinear methods of analysis. The products of the analysis are a conditional probability, $p(EDP \mid IM)$, for each *EDP*, which are integrated over the $p\langle IM \rangle$ to estimate the mean annual probability of exceedance of each *EDP*, $p\langle EDP \rangle$. The third step in the process is to relate the *EDP*s to Damage Measures (*DM*s) that describe the physical state of the building. The outcome of this step are conditional probabilities, $p(DM \mid EDP)$, which can be integrated over the $p\langle EDP \rangle$ to calculate the mean annual probability of exceedance for the *DM*, $p\langle DM \rangle$. The fourth and final step in the PEER_ATC-58 methodology is to calculate Decision Variables (*DV*s). The mean annual probability of exceeding a *DV*, $p\langle DV \rangle$, is calculated by integrating the conditional probability $p(DV \mid DM)$ (or loss function) over the $p\langle DM \rangle$. The PEER_ATC-58 methodology can be expressed in terms of a triple integral of (2) based on the total probability theorem, namely,

$$v(DV) = \iiint G\langle DV \big| DM \rangle \mid dG\langle DM \big| EDP \rangle \mid dG\langle EDP \big| IM \rangle \mid d\lambda(IM) \tag{2}$$

where all terms have been defined previously. Column 2 of Table 2 below lists *IM*s, *EDP*s, *DM*s and *DV*s that could be adopted by the ATC-58 project for steel moment-frame construction. Column 3 lists similar measures that could be applied in the case of blast engineering.

	Earthquake engineering	Blast engineering
Intensity Measures	Peak ground acceleration Spectral acceleration at T_1 Spectral acceleration at T_1 and T_2	Charge weight and standoff Charge weight and location
Engineering Demand Parameters[1]	Demand-to-capacity ratios Beam plastic rotation Beam shear Column axial load, moment, shear Column plastic rotation Inter-story drift	Demand-to-capacity ratios Beam plastic rotation Beam shear, axial load Column axial load, moment, shear Column plastic rotation Floor vertical displacement
Damage Measures	Deaths Dollars Downtime	Deaths Dollars Downtime
Decision Variables	Annualized loss Performance objective	Performance objective

1. For steel moment-frame construction only

Table 2. Sample *IM*s, *EDP*s, *DM*s and *DV*s for performance-based engineering

PERFORMANCE-BASED BLAST ENGINEERING

Introduction

Prior to the mid-1990s, analysis and design of building structures in the United States to resist the effects of blast loading and progressive collapse was undertaken by a relatively small group of specialty design professional consultants for a limited number of clients that managed high-exposure facilities such as government buildings, courthouses, and defense- and energy-related structures. Mainstream structural-engineering consultancies were not involved in blast engineering. The terrorist attacks on the World Trade Center in 1993 and 2001, the Murrah Building in 1995, and the Pentagon in 2001 altered substantially the attitude of the structural engineering community, building owners and insurers regarding blast design of commercial building construction, and there is renewed design-professional interest in blast engineering. However, because the blast-engineering design-professional community is smaller than the earthquake community and blast considerations in *commercial* building design were the exception rather than the norm, there has been no national effort, on the scale of the FEMA-BSSC effort for earthquake engineering (FEMA 2000b), to produce guidelines and commentary for the analysis and design of blast- and progressive-collapse-resistant buildings.

The General Services Administration (GSA) has developed guidelines for progressive collapse analysis and design for new federal office buildings and major modernization projects (GSA 2003) but these guidelines are for limited distribution at the time of this writing. The GSA guidelines represent the state-of-the-practice in blast engineering of buildings but, similar to current building codes for seismic design, make use of indirect methods of analysis and prescriptive procedures of unknown reliability (Hamburger and Whittaker 2003). Resource documents for blast engineering are being developed currently by FEMA (FEMA 2004a, 2004b) but these documents will not provide explicit guidelines for analysis and design.

Performance-based blast engineering should make possible a process that permits design and construction of buildings with a realistic and reliable understanding of the risk of loss (physical, direct economic and indirect economic) that might occur as a result of future attack. This process could be used to (a) predict the global response, degree of damage (and perhaps economic loss) to a building subjected to a scenario blast event (Figure 6a) or physical attack (Figure 6b); (b) design individual buildings that are more loss-resistant than typical buildings designed using prescriptive criteria of a documents similar to GSA (2003); (c) design individual buildings with a higher confidence that they will actually be able to perform as intended for a blast attack; (d) design individual buildings that are capable of meeting the performance intent of the prescriptive criteria, but at lower construction cost than would be possible using the prescriptive criteria; (e) design individual buildings that are capable of meeting the performance intent of the prescriptive criteria, but which do not comply with all of the limitations of the prescriptive criteria with regard to configuration, materials and systems; (f) investigate the performance of typical buildings designed using prescriptive provisions and develop judgments as to the adequacy of this performance; and (g) formulate improvements to the prescriptive provisions so that more consistent and reliable performance is attained by buildings designed using prescriptive provisions.

Towards performance-based blast engineering

Components of equation (2) are broadly applicable to performance-based engineering for all loading conditions, normal and extreme. Significant overlaps should exist for extreme blast and earthquake loadings because inelastic response of the framing system is anticipated in both cases. That said, there are significant differences between blast and earthquake engineering in the loading environment (hazard or *IM*s) and important differences in simulation procedures and component response (*EDP*) assessment.

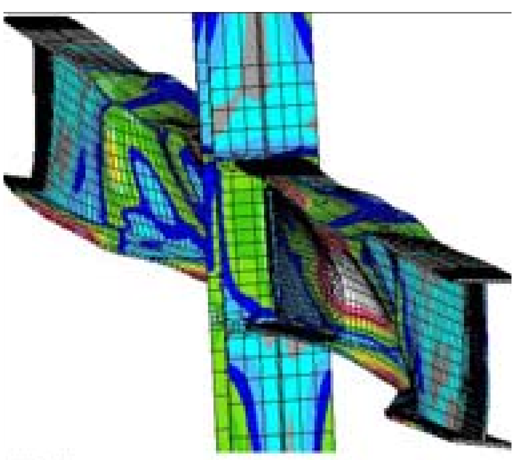
Blast loads on building structures (Biggs 1964; Mays and Smith 1995; Conrath et al. 1999) produce fundamentally different component responses than earthquake shaking. Further, blast loads are characterized deterministically at this time using scenario events (α charge weight at β distance) and not probabilistically using a hazard curve as described in the previous section. Using the terminology associated with equation (2), sample *IM*s for blast loading are listed in Table 2 above for explosives placed outside and inside a building. Similar to the translation of earthquake *IM*s into earthquake histories for the purpose of simulation, blast *IM*s must be transformed into loading functions, including pressure-impulse curves for assessing component integrity (response) to direct air-blast and the likelihood of component loss; and loading functions associated with component elimination due to air blast.

a. Murrah Federal Building

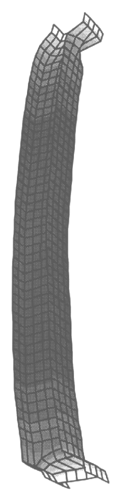
b. Deutsche Bank building (Berman et al. 2002)

Figure 6. Global response of buildings to blast loadings and physical attack

Simulation of building response to earthquake shaking involves subjecting a (nonlinear) mathematical model of the building frame to one or more earthquake histories. Nonlinear component models for such simulation should be based on experimental test data similar to that collected systematically in the SAC Steel Project (FEMA 2000c). Two assumptions made in the SAC Steel testing program and for the development of beam-component models were that beams are deformed primarily about their strong axis and twisting and distortion of the gross section is avoided. Both assumptions are reasonable for components in building frames subjected to earthquake shaking. However, neither assumption is valid for components in the immediate vicinity of air blast because such pressure loadings can produce gross damage and failure as shown in the numerical simulations of Figure 7. Further, the component models for earthquake simulation are based primarily on cyclic testing in the absence of significant axial load: testing conditions that are clearly inappropriate for components resisting progressive collapse.



a. W-shape beam cross section (*after Karns*)



b. W-shape beam web (*after Crawford*)

Figure 7. Gross distortion of steel components due to direct air blast

The *EDP*s of Table 2 for performance-based blast engineering are virtually identical to those for earthquake engineering. Demand-to-capacity (D/C) ratios are useful when linear methods of analysis are employed but calibration of D/C ratios to Damage Measures (*DM*s) using nonlinear response-history simulation is required. Values for the remaining *EDP*s could be output by nonlinear response simulations to develop *DM*s. Different conditional probabilities $p(DM \mid EDP)$ will result from air-blast and progressive collapse type loadings. Much full-scale experimental testing will be required to both facilitate such calculations of conditional probabilities and calibrate existing component models for blast and progressive-collapse analysis (Crawford et al. 2001).

Decision Variables (*DV*s) in the form of performance objectives have been identified for use in performance-based earthquake engineering. Details are provided in Table 1. Table 3 provides similar information for performance-based blast engineering. The proposed performance levels, damage descriptions and downtime estimates are preliminary and mutable, and are presented only to kindle discussion on *DV*s for performance-based blast engineering.

Performance level	*Damage description*	*Downtime*
Immediate occupancy	Negligible structural and nonstructural damage	24 hours
Life safety	Nonstructural and glazing damage; probable structural damage to beams and columns over a limited area; no collapse; adequate emergency egress; no loss of life due to structural damage	Several months to a year
Collapse prevention	Severe structural and nonstructural damage; structural damage over a wide area; incipient collapse; possible restricted egress; minimal loss of life due to structural damage	Possible total loss

Table 3. Possible building performance levels for blast-type loadings

SUMMARY REMARKS

Extreme events such as blast loadings and severe earthquake shaking will generally induce nonlinear behavior in building frames and produce substantial nonstructural damage. Although the current prescriptive procedures for design against blast and earthquake loadings might produce buildings of acceptable safety, the procedures are indirect, of unknown reliability, and might result in inefficient and costly construction. Performance-based engineering should facilitate design and construction of buildings with a realistic and reliable understanding of the risk of loss (physical, direct economic and indirect economic) that might occur as a result of future blast attack, earthquake shaking (or both).

The second-generation performance-based earthquake engineering methodology, which is being adopted for the ATC-58 project, is applicable conceptually to performance-based blast engineering. Sample intensity measures, engineering demand parameters, and performance levels for use in performance-based blast engineering were presented to foster discussion. Key similarities and differences between performance approaches for blast and earthquake engineering were identified.

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AISC RESEARCH ON STRUCTURAL STEEL TO RESIST BLAST AND PROGRESSIVE COLLAPSE

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Dr. Krauthammer is currently Professor of Civil Engineering at Penn State University, and Director of its Protective Technology Center. His main research and technical activities are directed at structural behavior under severe dynamic loads, including considerations of both survivability and fragility aspects of facilities subjected to blast, shock impact, and vibrations. Dr. Krauthammer is a Fellow of the American Concrete Institute (ACI), a member of the American Society of Civil Engineers (ASCE), and he serves on nine technical committees of ASCE, ACI, and the American Institute of Steel Construction (AISC). Dr. Krauthammer chairs the ASCE Task Committee on Structural Design for Physical Security. His research has been supported by various government agencies in the USA and abroad, and has written more than 300 research publications. He is a consultant to industry and government agencies in the USA and abroad, and has been invited to lecture in the USA and internationally.

ABSTRACT

This paper provides an overview of key issues related to the survivability of steel buildings subjected to explosive load incidents, and an outline of required research to address some of the problems that were identified in previous studies. Explosive loads associated with high explosive devices are expected to induce significant localized structural damage that could evolve into massive structural collapse. Recent numerically simulated responses of individual structural steel elements and connections to such loads have raised serious concerns about their ability to survive explosive loading incidents. Blast resistant structural systems are designed according to various guidelines, some of which are based on simplified assumptions whose suitability might be questioned. Furthermore, the relationships between localized structural damage and numerically-simulated progressive collapse have highlighted very complicated nonlinear dynamic phenomena. These phenomena require further investigation using more realistic representations of the corresponding issues.

INTRODUCTION AND BACKGROUND

Explosive loading incidents have become a serious problem that must be addressed quite frequently. Many buildings that could be loaded by explosive incidents are moment resistant steel frame structures, and their behavior under blast loads is of great interest. Besides the immediate and localized blast effects, one must consider the serious consequences associated with progressive collapse that could affect people and property in an entire building. Progressive collapse occurs when a structure has its loading pattern, or boundary conditions, changed such that structural elements are loaded beyond their capacity and fail. The residual structure is forced to seek alternative load paths to redistribute the load applied to it. As a result, other elements may fail, causing further load redistribution. The process will continue until the structure can find equilibrium either by shedding load, as a by-product of other elements failing, or by finding stable alternative load paths. In the past, structures designed to withstand normal load conditions were over designed, and have usually been capable of tolerating some abnormal loads. Modern building design and construction practices enabled one to build lighter and more optimized structural systems with considerably lower over design characteristics. Progressive collapse became an issue following the Ronan Point incident (HMSO, 1968), when a gas explosion in a kitchen on the 18^{th} floor of a precast building caused extensive damage to the entire corner of that building, as shown in Figure 1. The failure investigation of that incident resulted in important changes in the UK building code (HMSO, 1976). It requires to provide a minimum level of strength to resist accidental abnormal loading by either comprehensive 'tying' of structural elements, or (if tying is not possible) to enable the 'bridging' of loads over the damaged area (the smaller of 15% of the story area, or 70 m^2) , or (if bridging is not possible) to insure that key elements can resist 34 kN/m^2. These guidelines have been incorporated in subsequent British Standards (e.g., HMSO 1991, BSI 1996, BSI 2000, etc.). Although many in the UK attribute the very good performance of numerous buildings subjected to blast loads to these guidelines, it might not be always possible to quantify how close those buildings were to progressive collapse.

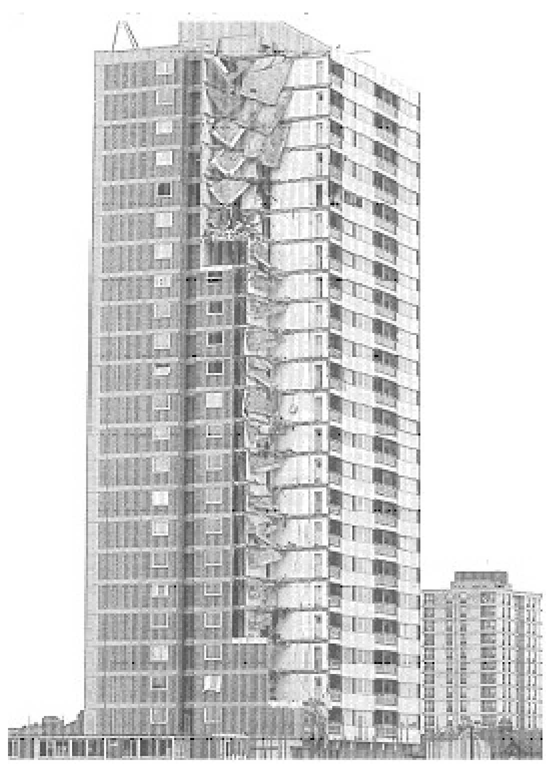


Figure 1 Post-Incident View of the Ronan Point Building (HMSO, 1968)

Recent developments in the efficient use of building materials, innovative framing systems, and refinements in analysis techniques could result in structures with lower safety margins. Both the Department of Defense (DoD) and the General

Services Administration (GSA) have issued clear guidelines to address this critical problem (DoD 2002, GSA 2003). Nevertheless, these procedures contain assumptions that may not reflect accurately the actual post attack conditions of a damaged structure, as shown in Figures 2a and 2b, that highlight the very complicated state of damage that must be assessed before the correct conditions can be determined. The structural behavior associated with such incidents involves highly nonlinear processes in both the geometric and material domains. One must understand that various important factors can affect the behavior and failure process in a building subjected to an explosive loading event, but these cannot be easily assessed. The idea that one might consider the immaculate removal of a column as a damage scenario, while leaving the rest of the building undamaged, is unrealistic. An explosive loading event near a building will cause extensive localized damage (e.g., terrorist attacks in London, Oklahoma City, etc.), affecting more than a single column. The remaining damaged structure is expected to behave very differently from the ideal situation. Therefore, it is critical to assess accurately the post attack behavior of structural elements that were not removed from the building by the blast loads in their corresponding damaged states. This requires one to perform first a fully-nonlinear blast-structure interaction analysis, determine the state of the structural system at the end of this transient phase, and then to proceed with a fully-nonlinear dynamic analysis for the damaged structure subjected to only gravity loads. Such comprehensive analyses are very complicated, they are very time consuming and require extensive resources, and they are not suitable for design office environments.

(a) General View

(b) Close up View

Figure 2 Post-Incident View of Building Damage from the 1992 St. Mary's Axe Bombing Incident in London

Damaged structures may have insufficient reserve capacities to accommodate abnormal load conditions (Taylor 1975, and Gross 1983). So far, there are few numerical examples of computational schemes to analyze progressive collapse. Typical finite element codes can only be used after complicated source level modification to simulate dynamic collapse problems that contain strong nonlinearities and discontinuities. Several approaches have been proposed for including progressive collapse resistance in building design. The alternative load path method is a known analytical approach that follows the definition of progressive collapse (Yokel et al. 1989). It refers to the removal of elements that failed the stress or strain limit criteria. In spite of its analytical characteristics, alternative load path methods are based on static considerations, and they may not been be adequate for simulating progressive collapse behavior. Choi and Krauthammer (2003) described an innovative approach to address such problems by using algorithms for external criteria screening (ECS) techniques applicable to these types of problems. As a part of such ECS, element elimination methods were classified into direct and indirect approaches, and compared with each other. A variable boundary condition (VBC) technique was also proposed to avoid computational instability that could occur while applying the developed procedure. An effective combination of theories was established to analyze progressive collapse of a multi-story steel frame structure. For illustration, the procedure was applied to a two-dimensional steel frame model. Stress/strain failure criteria of linear, elastic-perfectly plastic, and elastic-plastic with kinematic hardening models were considered separately. A buckling failure criterion was also considered to supplement a strain failure criterion in the elastic-perfectly plastic model. That approach is currently being developed further to study the physical phenomena associated with progressive collapse, and to develop fast running computational algorithms for the expedient assessment of various multi-story building systems.

The 1994 Northridge earthquake highlighted troublesome weaknesses in design and construction technologies of welded connections in moment-resisting structural steel frames. As a result, the US steel construction community embarked on an extensive R&D effort to remedy the observed deficiencies. During about the same period, domestic and international terrorist attacks have become critical issues that must be addressed by structural engineers. Here, too, it has been shown that structural detailing played a very significant role during a building's response to blast. In blast resistant design, however, most of the attention during the last half century has been devoted to structural concrete. Since many buildings that could be targeted by terrorists are moment-resisting steel frames, their behavior under blast is of great interest, with special attention to structural details. Typical structural steel welded connection details, currently recommended for earthquake conditions, underwent preliminary assessments for their performance under blast effects. The assessments also addressed current blast design procedures to determine their applicability for both the design and analysis of such details. The finding highlighted important concerns about the blast resistance of structural steel details, and about the assumed safety in using current blast design procedures for structural steel details.

Obviously, one must address not only the localized effects of blast loads, and the idealized behavior of typical structural elements (e.g., columns, girders, etc.), but also the behavior of structural connections and adjacent elements that define the support conditions of a structural element under consideration. The nature of blast loads, the behavior of structural connections under such conditions, and progressive collapse are addressed in the following sections to provide the background for current and proposed research activities.

BLAST AND RELATED EFFECTS

Typical Blast Effects

Blast effects are associated with either nuclear or conventional explosive devices. Although small nuclear devices (e.g., tactical size) could be used by terrorists, the associated technical problems include many serious issues that could be far more complicated to address than blast effects on buildings. Therefore, nuclear weapon effects are not addressed in this paper. The interested reader can find useful information on this topic in other sources (e.g., ASCE 1985). Similarly, the effects of some industrial explosions are described elsewhere (ASCE, 1997). Scaling laws are used to predict the properties of blast waves from large explosive devices based on test data with much smaller charges (Johansson and Persson 1970, Baker 1973, Baker et al. 1983). The most common form of blast scaling is the Hopkinson-Cranz, or cube-root scaling (Hopkinson 1915, Cranz 1926). It states that self-similar blast waves are produced at identical scaled distances when two explosive charges of similar geometry and of the same explosive, but of different sizes, are detonated in the same atmospheric conditions. It is customary to use as a scaled distance a dimensional parameter, Z, as follows:

$$Z = R/E^{1/3}, \text{ or } Z = R/W^{1/3} \tag{1}$$

where R is the distance from the center of the explosive source, E is the total explosive energy released by the detonation (represented by the heat of detonation of the explosive, H), and W is the total weight of a standard explosive, such as TNT, that can represent the explosive energy. Blast data at a distance *R* from the center of an explosive source of characteristic dimension d will be subjected to a blast wave with amplitude of P, duration t_d, and a characteristic time history. The integral of the pressure-time history is defined as the impulse I. The Hopkinson-Cranz scaling law then states that such data at a distance ZR from the center of a similar explosive source of characteristic dimension Zd detonated in the same atmosphere will define a blast wave of similar form with amplitude P, duration Zt_d and impulse ZI. All characteristic times are scaled by the same factor as the length scale factor Z. In Hopkinson-Cranz scaling, pressures, temperatures, densities, and velocities are unchanged at homologous times. The Hopkinson-Cranz scaling law has been thoroughly verified by many experiments conducted over a large range of explosive charge energies. Limited reflected impulse measurements (Huffington and Ewing 1985) showed that Hopkinson-Cranz scaling may become inapplicable for close-in detonations, e.g., $Z < 0.4$ ft/lb$^{1/3}$ (0.16 m/kg$^{1/3}$).

The character of the blast waves from condensed high explosives is remarkably similar to those of TNT, and these curves can be used for other explosives by calculating an equivalent charge weight of the explosive required to produce the same effect as a spherical TNT explosive. Generally, the equivalent weight factors found by comparing airblast data from different high explosives vary little with scaled distance, and also vary little dependent on whether peak overpressure or side-on impulse is used for the comparisons. When actual comparative blast data exist, its average value can be used to determine a single number for TNT equivalence. When no such data exist, comparative values of heats of detonation, H, for TNT and the explosive in question can be used to predict TNT equivalence (Department of the Army, Navy, and Air Force 1990, U.S. Department of Energy 1992, Conrath et al. 1999).

The theoretical heats of detonation for many of the more commonly used explosives are listed in various sources (e.g., Baker et al. 1983, Appendix A of U.S. Department of Energy 1992), along with TNT equivalency factors (Department of the Army 1986, and Department of the Army, Navy, and Air Force 1990). This method of computing TNT equivalency is related primarily to the shock wave effects of open-air detonations, either free-air or ground bursts. Limitations of this approach have been discussed in various publications (e.g., Conrath et al. 1999). Typical sources of compiled data for airblast waves from high explosives are for spherical TNT explosive charges detonated at standard sea level. The data are scaled according to the Hopkinson-Cranz (or cube-root) scaling law.

Unconfined Explosions

A typical representation of a blast-induced pressure-time history curve is shown in Figure 3. One can note that the pressure rises sharply above atmospheric levels upon the arrival of the shock wave, then it decays exponentially. During this decay, the pressure will decrease below atmospheric levels, but then recover to it. The phase during which the pressure is above the atmospheric level is termed “positive phase”, while it is known as the “negative phase” for the duration it is below the atmospheric level.

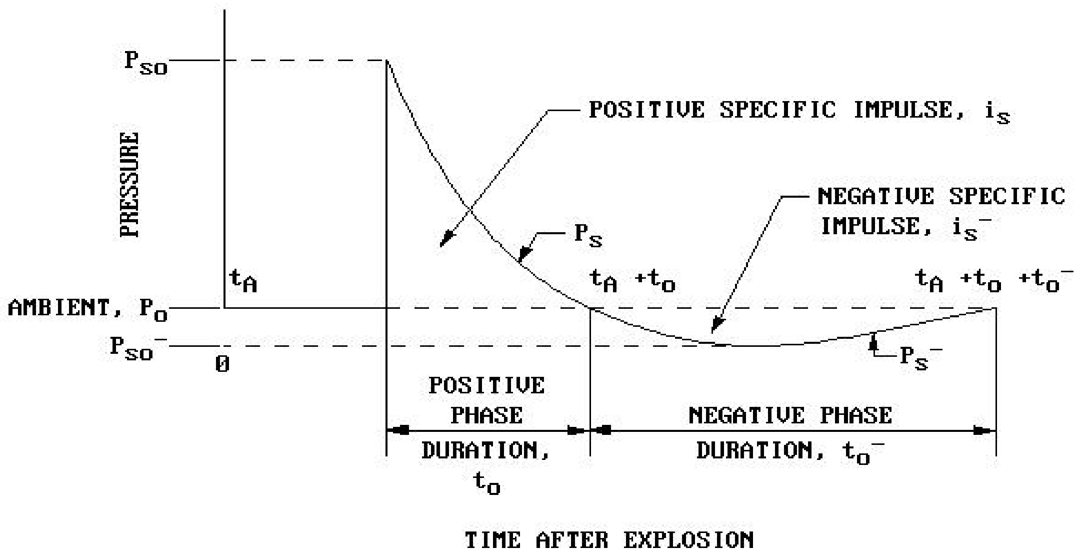


Figure 3 Typical Blast-Induced Pressure-Time history (TM 5-1300, 1990)

An acceptable set of standard airblast curves for the positive-phase blast parameters is shown in various references (e.g., Kingery and Bulmash 1984, Department of the Army 1986, Departments of the Army, Navy, and Air Force 1990). The procedures in (Department of the Army, 1986) have been implemented in the computer code ConWep (Hyde 1993) that can be used for calculating a wide range of weapon and explosive effects. These sources present the scaled form of various blast wave parameters, e.g., the peak side-on overpressure, P_s (psi), side-on specific impulse, i_s (psi-ms), shock arrival time, t_a (ms), positive phase duration, t_d (ms), peak normally reflected overpressure, P_r (psi), normally reflected specific impulse, i_r (psi-ms), shock front velocity, U (ft/ms), wave length of positive phase, L_W (ft). The normally reflected pressure and impulse are greater than the corresponding side-on values because of the pressure enhancement caused by arresting flow behind the reflected shock wave. Various sources (e.g., U.S. Department of Energy 1992) present methodologies for calculating such parameters. For an explosive charge detonated on the ground surface, one can use the free-air blast curves to determine blast wave parameters by adjusting the charge weight in the ground burst to account for the enhancement from the ground reflection. For a perfect reflecting surface, the explosive weight is simply doubled. When significant cratering takes place, a reflection factor of 1.8 is more realistic. This simple approach is recommended for an explosion at or very near the ground surface. This approach is still valid. However, test data are available and have been compiled from tests using hemispherical TNT charges on the ground surface. From these data, blast curves for the positive-phase blast parameters were developed and are widely used as the standard for ground bursts. It is assumed that structures subjected to the explosive output of a surface burst will usually be located in the pressure range where the plane wave concept can be applied. Therefore, for a surface burst, the blast loads acting on structure surface are calculated as described for an air burst except that the incident pressures and other positive-phase parameters of the free-field shock environment are obtained from the appropriate charts. For the normally reflected parameters, the structural element would be perpendicular to the direction of the shock wave. Otherwise, the wave will strike the structure at an oblique angle.

The simplest case of blast wave reflection is that of normal reflection of a plane shock wave from a plane, rigid surface. In this case, the incident wave moves at velocity U through still air at ambient conditions. The conditions immediately behind the shock front are those for the free-air shock wave. When the incident shock wave strikes the plane, rigid surface, it is reflected and moves away from the surface with a velocity U_r into the flow field and compressed region associated with the incident wave. In the reflection process, the incident particle velocity u_s is arrested ($u_s = 0$ at the reflecting surface), and the pressure, density, and temperature of the reflected wave are all increased above the values in the incident wave. When a plane wave strikes a structure at an angle of incidence, the oblique reflected pressures will be a function of the shock strength. Also, as a blast wave from a source some distance from the ground reflects from the ground, the angle of incidence must change from normal to oblique. Normally reflected blast wave properties usually provide upper limits to blast loads on structures. Nevertheless, one may have to consider cases of blast waves that also strike at oblique angles. The effects of the angle of incidence versus the peak reflected pressure $P_{r\alpha}$ are shown in the references cited previously. Accordingly, one can show the incident and reflected pressure pulses on the same time scale, as presented in Figure 4.

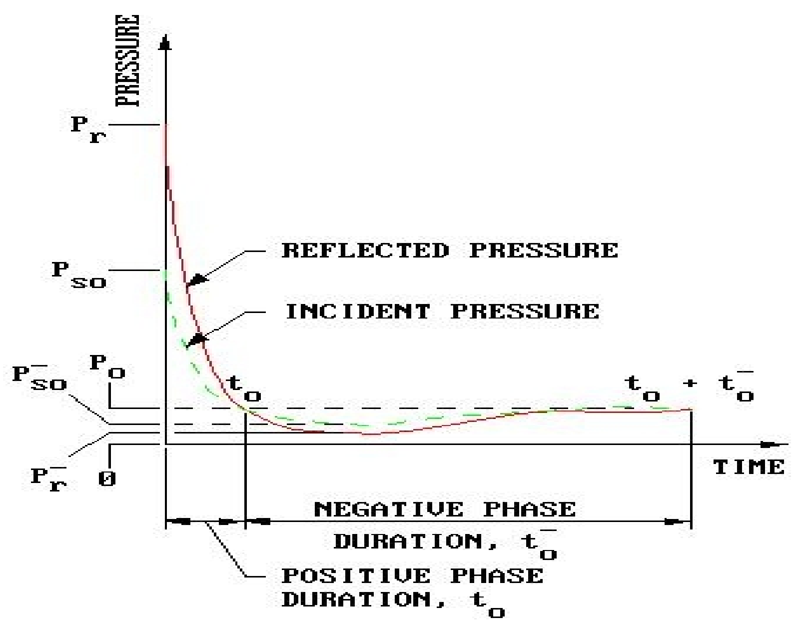


Figure 4 Incident and Reflected Pressure-Time Histories (TM 5-1300, 1990)

Confined Explosions

Confined and contained explosions within structures result in complicated pressure-time histories on the inside surfaces. Such loading cannot be predicted exactly, but approximations and model relationships exist to define blast loads with a good confidence. These include procedures for blast load determination due to initial and reflected shocks, quasi-static pressure, directional and uniform venting effects, and vent closure effects. The loading from a high-explosive detonation within a confined (vented) or contained (unvented) structure consists of two almost distinct phases. The first is the shock phase, where incident and reflected shocks inside structures consist of the initial high-pressure, short-duration reflected wave, and several later reflected shocks reverberations of the initial shock within the structure. The second is called the gas loading phase that attenuated in amplitude because of an irreversible thermodynamic process. These are complicated wave forms because of the involved reflection processes within the vented or unvented structure. The overpressure at the wall surface is termed the normally reflected overpressure, and is designated P_r. Following the initial internal blast loading, the shock waves reflected inward will usually strengthen, as they implode toward the center of the structure, and then attenuate, as they move through the air and re-reflect to load the structure again. The secondary shocks will usually be weaker than the initial pulse. The shock phase of the loading will end after several such reflection cycles.

SHOCK (NCEL, 1988) is a computer code for estimating internal shock loads. This code can be used to calculate the blast impulse and pressure on all or part of a cubicle surface bounded by one to four rigid reflecting surfaces. The code calculates the maximum average pressure on the blast surface from the incident and each reflected wave and the total average impulse from the sum of all the waves. The duration of this impulse is also calculated by assuming a linear decay from the peak pressure. This code is based on the procedures in TM 5-1300 (Departments of the Army, Navy, and Air Force 1990). Shock impulse and pressure are calculated for each grid point for the incident wave and for the shock reflecting off each adjacent surface. The program includes a reduced area option which allows determination of average shock impulse over a portion of the blast surface or at a single point on the surface. The code calculates blast parameters for scaled standoff distances ($R/W^{1/3}$) between 0.2 (0.079 m/kg$^{1/3}$) and 100.0 ft/lb$^{1/3}$(39.7 m/kg$^{1/3}$). The program, however, does not account for gas pressure load contributions. When an explosion from a high-explosive source occurs within a structure, the blast wave reflects from the inner surfaces of the structure, implodes toward the center, and re-reflects one or more times. The amplitude of the re-reflected waves usually decays with each reflection, and eventually the pressure settles to what is termed the gas pressure loading phase. Considering poorly vented or unvented chambers, the gas load duration can be much longer than the response time of the structure, appearing nearly static over the time to maximum response. Under this condition, the gas load is often referred to as a quasi-static load. For vented chambers, the gas pressure drops more quickly in time as a function of room volume, vent area, mass of vent panels, and energy release of the explosion. Depending on the response time of structural elements under consideration, it may not be considered quasi-static. The gas load starts at time zero and overlaps the shock load phase without adding to the shock load, as illustrated in Figure 5, where the shock phase and the gas phase are idealized as triangular pulses. The quasi-static portion of the pressure pulse can be obtained with the computer code FRANG (Wager and Connett, 1989) to predict of the load history and combining the two curves to form the complete pressure-time history. Noting that the shock and quasi-static pressures are not added where they overlap, but are merely intersected to define the load history, is important.

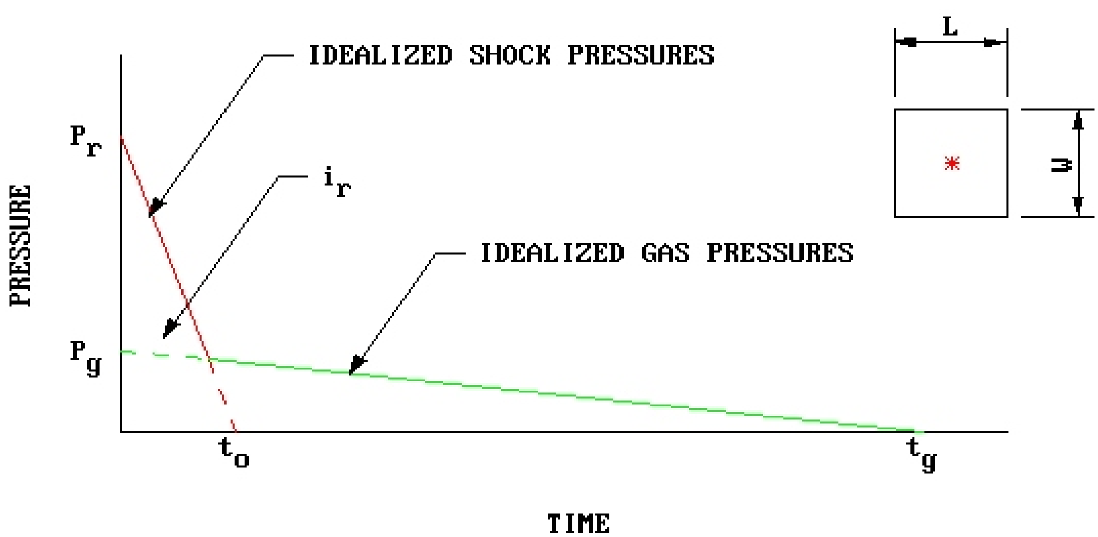


Figure 5 Typical Combined Shock and Gas Load in a Small Chamber (Departments of the Army, Navy, and Air Force 1990).

The code BLASTX (Britt 1992) treats the combined shock wave (including multiple reflections off walls) and explosive gas pressure produced by the detonation of a conventional high explosive in a closed or vented, rigid or responding walls that are allowed to fail under gas pressure loading in rectangular, cylindrical, or L-shaped rooms. The code allows the propagation of shocks and gas into adjacent rectangular or box-shaped spaces. The code does have the capability to treat multiple non simultaneous explosions in a room, modifications of shock arrival times and peak pressures to account for Mach stem effects, and the option to obtain pressure and impulse wave forms averaged over a number of target points on a wall. As with SHOCK, it does not account for movement of any of the walls or the roof, although recent versions of the code do allow openings to occur based on defined failure criteria and as created by combined shock and gas pressures. Although gas pressures are propagated through failed surfaces, shocks are not vented through failed openings.

Ballistic Attack, Fragmentation, and Ground Shock

Another class of threats is related to ballistic attack and fragmentation effects. Information on various weapon systems that could be used for such applications can be found in several sources (e.g., Department of the Army 1986, Department of the Army, Navy, and Air Force 1990, US Department of Energy 1992, Conrath at al. 1999). However, since these types of threat may not pose a direct hazard to cause massive structural failure, they are not addressed here. Usually, ground shock is also not a significant issue for terrorist incidents, since typical attacks involve above ground explosions. Nevertheless, one may have to consider ground shock for special cases (e.g., if a threat might include a buried charge). Therefore, these issues will not be addressed further in this paper, and the interested reader is referred to the sources cited above.

BLAST-RESISTANT STRUCTURAL STEEL CONNECTIONS AND PROGRESSIVE COLLAPSE

One of the important issues in structural behavior is the ability of connections to resist severe dynamic loads. Design guidelines for structural steel connections in the US (AISC, 1994) were developed based on experimental and theoretical investigations. However, the findings from the Northridge earthquake, and in recent reports (e.g., Bonowitz et al 1995, Engelhardt and Hussain 1993, Engelhardt et al. 1995) on damage to steel structures due to seismic loads suggested a surprisingly poor performance of their connections, as compared to expected behavior (AISC 1992). An extensive research program was undertaken to address the observed deficiencies (Engelhardt et al. 1995, Kaufman et al. 1996a,b, Richard et al. 1995, Tsai and Popov 1995, El-Tawil et al 2000, Stojadinović et al. 2000, Davis 2001), and important design modification were introduced (AISC 1997, FEMA 2000). Nevertheless, these modified connection details may exhibit similar deficiencies under blast effects. Therefore, it is very important to assess their behavior under blast effects, and to identify any possible behavioral difficulties. Structural behavior under dynamic loads requires attention to the relationship between the dynamic characteristics of the structure and the applied loads. Design specifications should address this relationship to insure that the various structural details are blast resistant. TM 5-1300 (Department of the Army, Navy, and Air Force, 1990)contains guidelines for the safe design of blast resistant steel connections. However, the adequacy of these design procedures is not well defined because of insufficient information about the behavior of the steel connections under blast loads.

High loading rates can influence the mechanical properties of structural materials (Department of the Army, Navy, and Air Force, 1990, and Soroushian and Choi 1987), and the use of dynamic increase factors (DIFs) for describing strain rate enhancement is well known. A DIF is the ratio of the strain rate enhanced strength to the static strength (e.g., the ratio of the dynamic and static yield stresses for a material). This effect of higher strain rates on the mechanical properties of steel is important for blast-resistant design. DIFs are used for both design and approximate analyses (e.g., single-degree-of-freedom calculations). Nevertheless, DIFs have to be used with care in advanced numerical simulations. It is well known that the steel yield stress is enhanced by strain rates while the ultimate stress is affected much less. The effects of high rate dynamic loadings on the structural responses were investigated in (Krauthammer et al. 2001, and 2002) by employing the recommended DIF values in TM 5-1300 for both the design and the numerical simulations. In those studies, typical modified structural steel connections for seismic conditions were assessed under explosive loads by employing the design procedure in TM 5-1300, and additional and empirical analysis tools. The maximum safe amount of the contained explosive charge and the blast resistant capacities of the connections were estimated. A reliable finite element code developed for simulating short-duration dynamic events, DYNA3D (Whirley and Engelmann, 1993), was validated and used to investigate the behavior of the structural steel connections under the expected blast loads. A hypothetical one-story frame structure with seismic resistant knee connections, assumed to be part of a multi-story building, was selected, as shown in Figure 6.

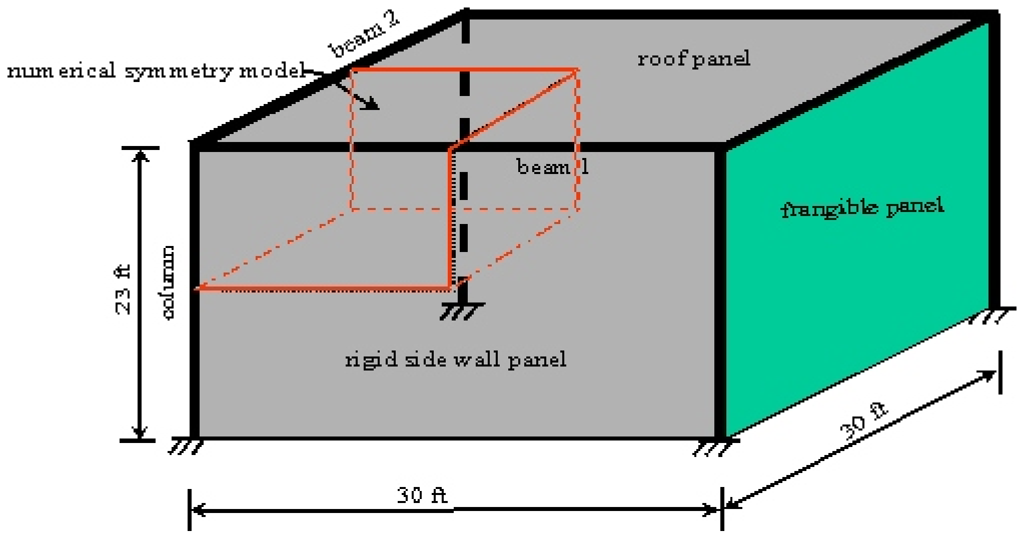
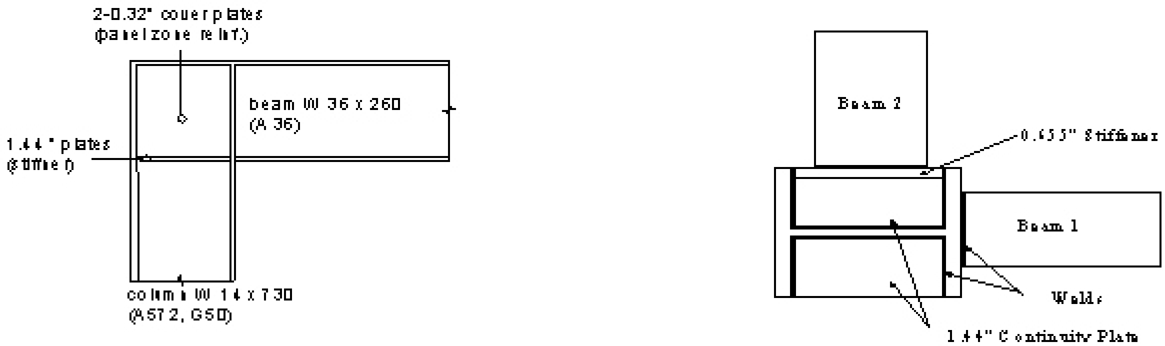


Figure 6 Hypothetical Building Model

TM 5-1300 provides methods of design for explosive safety. Also, the manual provides procedure for deriving the blast load parameters and for calculating the dynamic responses of structural members. The adopted details of the structural model were based on a typical seismic resistant steel knee connection at a corner of a frame. W-14x730 (A572 GR50) and W-36x260 (A36) were employed for the column and beam members, respectively. The weld was made with E70TG-K2 fluxed core electrodes that have a 70 ksi yield stress. The post-Northridge connection was reinforced with 36-mm-thick continuity plates between the column flanges and 25-mm-thick cover-plates on the beam flanges, to enhance the required moment resistance. The design procedures outlined in Section 5 of TM 5-1300 were used to evaluate the maximum blast load that can be applied to the beam and column members for the structure. The computer codes Shock and Frang were employed also for deriving by trial-and-error the weight of an equivalent TNT charge. One wall was assumed as a frangible panel (i.e., it was assumed to blow out and permit quick venting of the internal pressures). Finally, the expected structural deformations were computed based on the given load and the procedure outlined in TM 5-1300.

The internal blast loads were derived from the detonation of a TNT charge at the geometric center of the floor in the structures. According to TM 5-1300, a load pulse from an internal explosion can be represented by a short and intense shock pressure and a longer duration lower intensity gas pressure. The maximum rotational deformations at the connections are typically used to assess structural damage. These rotations were investigated to define the structural capacities of the connections under opening explosive loads (i.e., an internal explosion will cause the connections to open). Stresses and strains at critical points of welds and panel zones were checked to identify more detailed deformation mechanisms. Two types of steel connections were employed in this study, as shown in Figure 7.

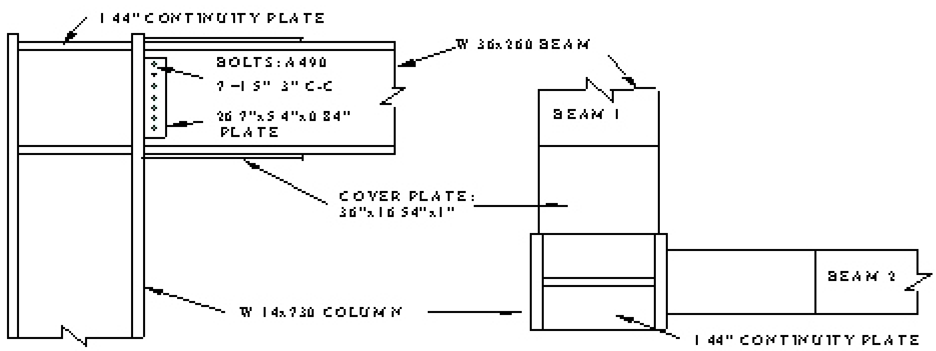
The first type of connection was used extensively before the Northridge earthquake, and the second was recommended for seismic applications after the studies of the Northridge earthquake. As a first step, the blast resistant capacity of the corner knee connection was analyzed according to TM 5-1300. The preliminary assessment provided an estimated maximum safe explosive charge weight (W equal to 24.7 lbs or 11.23 kg TNT) that the structure could be expected to resist without exceeding recommended damage levels. The rotational deformations were estimated at the connection, based on the computed structural response. Additionally, that analysis was used also to define the corresponding pressure-time histories that would be applied to the structure in the advanced numerical simulation. Finally, numerical simulations were conducted with DYNA3D for comparison with the findings based on TM 5-1300, and to obtain useful information about the possible responses of the steel connections under internal explosions. Since TM 5-1300 requires one to design a structure for 1.2 times the expected explosive weight, the simulation with DYNA3D were performed for blast loads from the detonation of a charge weight of W/1.2, leading to 20.6 lbs or 9.35 kg TNT. The analysis and design loads are described in Figure 8.



a. Side View

(a) Pre-Northridge Connection Model

b. Top View

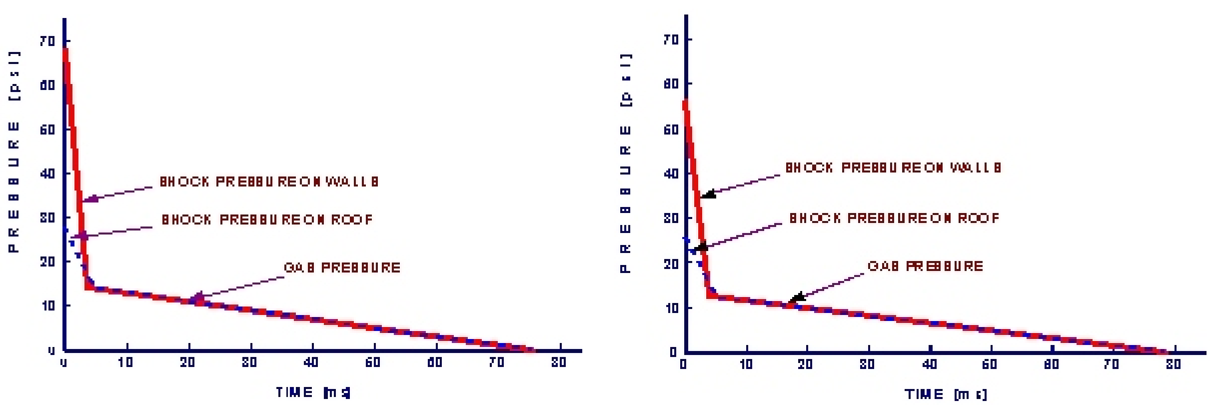


a. Side View

b. Top View

(b) Post-Northridge Connection Models

Figure 7 Design and Construction Details



a. Design Loads for 24.7 lbs (11.23 kg) TNT.

b. Analysis Loads for 20.6 lbs (9.35 kg) TNT.

Figure 8 Blast Loads

The maximum rotational deformations at the connections were investigated to define the structural capacities of the connections under opening explosive loads. Stresses and strains at the critical points of welds and panel zones were checked to identify the more detailed deformation mechanism. These connections between W-shape cross-sections were studied in both two-dimensional (2D) and three-dimensional (3D) frame structures. They were developed to obtain preliminary general information of the responses of the steel connections under the blast loads, and for initial comparisons with the results of the approximate analyses. The 3D frame models had 7,652 3-D elements. They were developed to analyze the three-dimensional behavior of the system, and the pressure loads on wall and roof panels were transmitted to the frame members. To investigate the strain rate effect on the steel properties and structural responses, the numerical simulations were done with and without consideration of Dynamic Increasing Factors (DIF). Additionally, since this structural model could be part of a multi story building, the effects of gravity dead loads were considered. These equivalent gravity loads were represented by a 206.9 MPa axial dead load on the columns and 58.4 kN/m vertical dead load on the beams. Each case was studied with and without DIFs, and with and without dead loads. All the numerical simulations were based on combining DIF values and validation against precision test data, and only the results for the 3D connections are summarized in Tables 1 and 2, and Figures 9 and 10 .

Table 1 Computed Peak Motions, Strains and Stresses (Combined Shock and Gas Pressure Effects
(All data are for beam members, for which f_y = 36 ksi, and f_{yd} = 46.4 ksi)

Case		Max. Strs. (ksi)	Max. Strn. (in./in.)	Max. Disp. (in.)	Max. Vel. (in./ms)	Max. Acc. (in./ms^2)
Pre Northridge	3D	49.5*	0.206*	27.6**	842.1**	16E6**
	DL-3D	47.4*	0.14*	27.3**	812.9**	48E5**
	D-3D	67.5*	0.27*	19.8**	624.0**	11E6**
	DL-D-3D	69.0*	0.15*	19.3**	637.0**	21E6**
Post Northridge	3D	58.0*	0.079	2.7	129.7	62000000
	DL-3D	51.5*	0.09	17.2	1301	11170000
	D-3D	71*	0.059	2	440.1	18500000
	DL-D-3D	62*	0.058	0.59	1187	4700000

* Stress and/or strain data indicated possible failure, ** Last useful data before failure state.

That preliminary study produced findings that raised major concerns about the blast resistance of moment-resisting structural steel connections, and the safety of using TM 5-1300 for the design of structural steel connections. It was shown that moment-resisting structural steel welded connections subjected to 'safe' explosive loads may fail due to weld fracture. Furthermore, it was shown that the corresponding deformations of the designed structural elements may exceed the limits set in TM 5-1300. It was shown further that dead loads have an adverse effect on the behavior. This was due to the added bending and twisting of the beams once they were deformed by the blast effects. Since TM 5-1300 does not address such effects, current design procedures should be modified to reflect the structural damage caused by weak axis deformations. Although steel is not expected to be very sensitive to strain rate effects, as compared to concrete, serious strain rate effects were note in the structural steel connections. Nevertheless, it was observed that the current DIFs, as defined in TM 5-1300, need to be modified to address more detailed pressure levels and the differences between 2-D and 3-D behavior. It is recommended that more detailed DIF applications, associated with pressure levels, might be necessary to avoid possible overestimation of strain rate effects. This could be very important when 2-D analysis are used to design 3-D structures.

It was concluded that improved design approaches for structural steel welded connections in blast resistant buildings are urgently needed. Such design approaches should be derived based on additional studies that must be supported by combined theoretical, numerical, and experimental efforts. On going studies at Penn State are aimed at addressing the issues raised during this investigation, but they need to be expanded significantly, in cooperation with several DoD organizations.

Table 2 Observed End Rotations

CASE		RESPONSE	MAX SUPPORT ROTATION (θ°)		
			BEAM		COLUMN
			X	Y	STRONG AXIS
ALLOWABLE (TM 5-1300)		FLEXURE	2		
Pre Northridge	3D	FLEXURE	FAILED*	FAILED*	FAILED*
	D-3D	FLEXURE	FAILED*	FAILED*	FAILED*
	DL-3D	FLEXURE	FAILED*	FAILED*	FAILED*
	DL-D-3D	FLEXURE	FAILED*	FAILED*	FAILED*
Post Northridge	3D	FLEXURE	7.26**	8.24**	0.33
	D-3D	FLEXURE	5.58**	5.31**	0.26
	DL-3D	FLEXURE	7.79**	11.3**	18.32**
	DL-D-3D	FLEXURE	3.97	3.07	0.93

* Stress and/or strain data indicated failure at and near the beam-column interface (see Table 1, and Figures 4 and 5). The, beams separated from column, the simulation became unstable and had to be terminated.** Very large rotations, 5°-12° indicate severe damage (TM 5-1300).

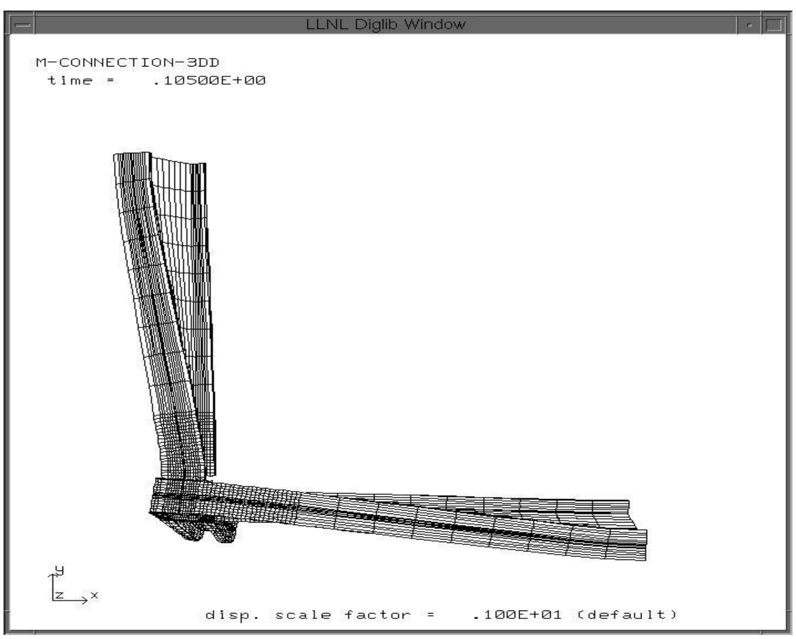


Figure 9 Deformation Shape of the Pre-Northridge Case DL-3D.

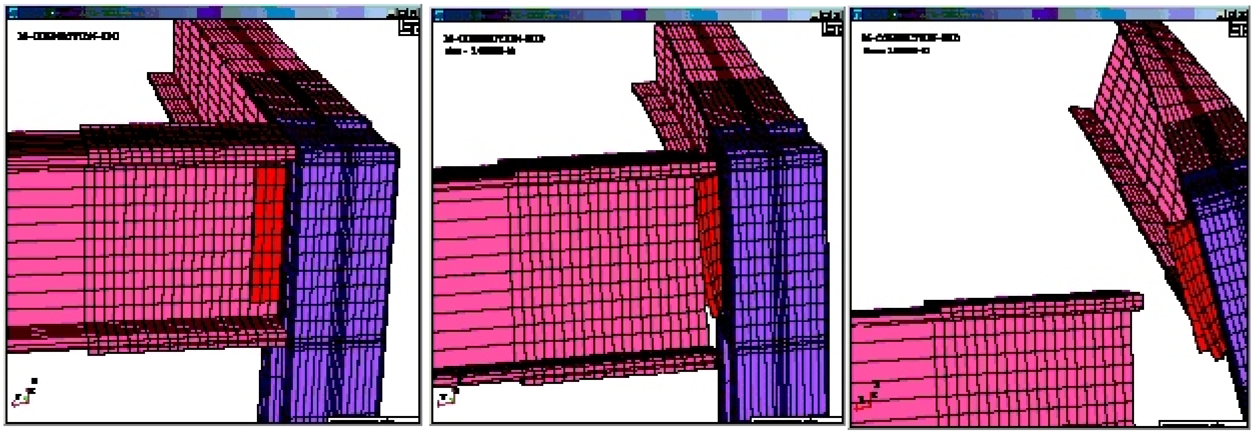


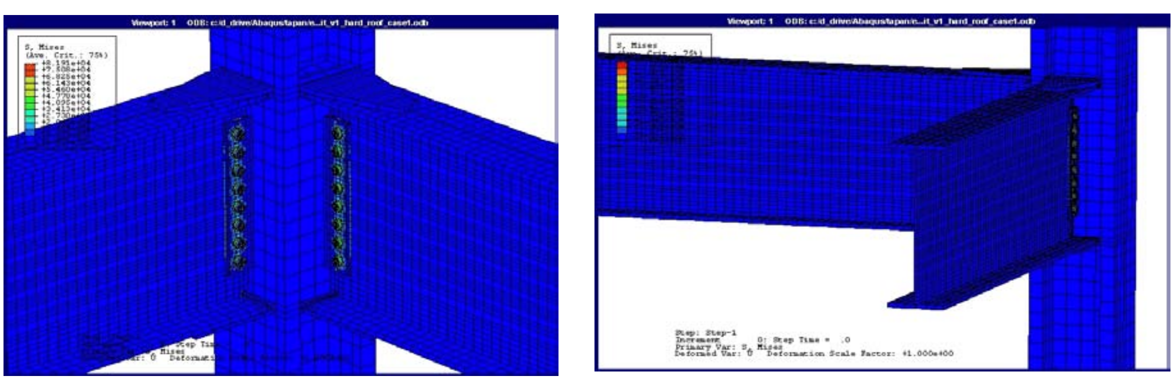
Figure 10 Deformation Sequence of the Post-Northridge Case DL-3D.

PROPOSED FOLLOW UP STUDIES

The planned follow up studies will be aimed at addressing the following issues (Mosher et al., 2001, and Krauthammer January and October 2003):

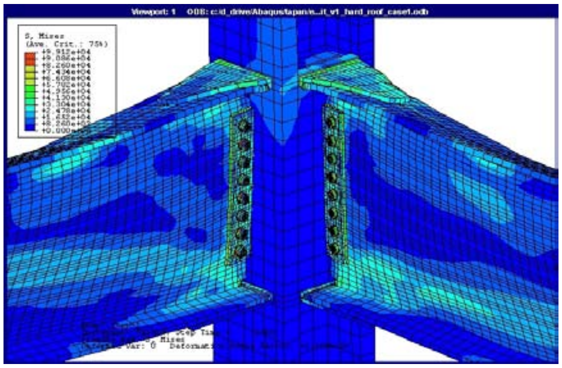
- A comprehensive assessment of existing and modified structural steel elements and connection under blast and impact loads.
- Understanding the physical phenomena that could cause progressive collapse and/or are associated with progressive collapse.
- Defining the relationships between localized damage in structural steel buildings and progressive collapse.
- Development of improved design guidelines to enhance the survivability of structural steel building under blast and impact loading environments.

The first two topics are currently under investigation at Penn State, as a follow up to previous activities described above. In the study of structural steel connections, the same type of connections described above have been modeled in great detail to capture the role of various parameters in the behavior (e.g., columns, girders, continuity and/or cover plates, bolts, welds, etc.). One such numerical model is shown in Figure 11.

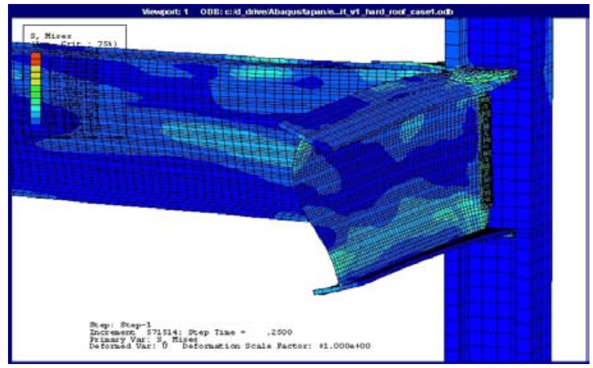


a. Model View 1   *b. Model View 2*

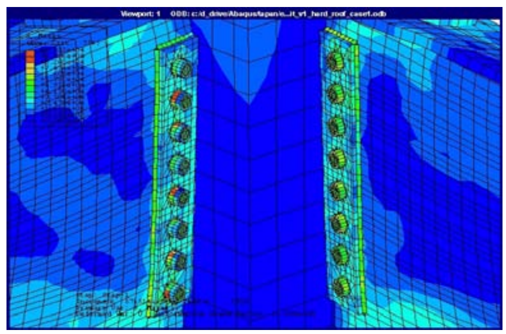
Figure 11 Detailed Structural Steel Connection Model



c. Deformed Shape



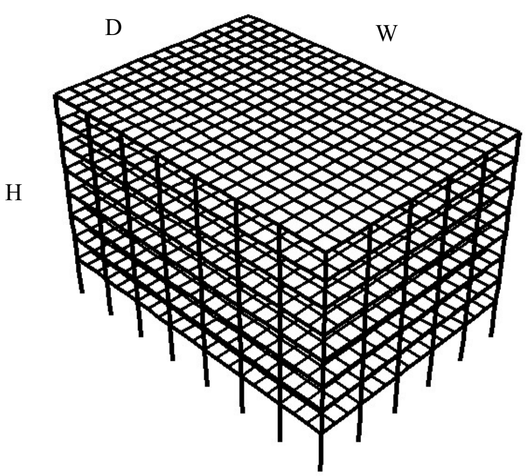
d. Deformed Shape



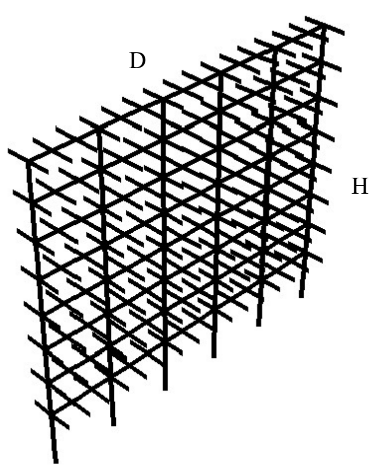
e. Shear Tab-Bolt Details

Figure 11 Detailed Structural Steel Connection Model (cont.)

The study on progressive collapse is focused on the behavior of individual elements, their support conditions, and their performance in a multi story moment resisting frame (115'8" H X 175' W X 125' D), as shown in Figure 12.



a. Original three dimensional model



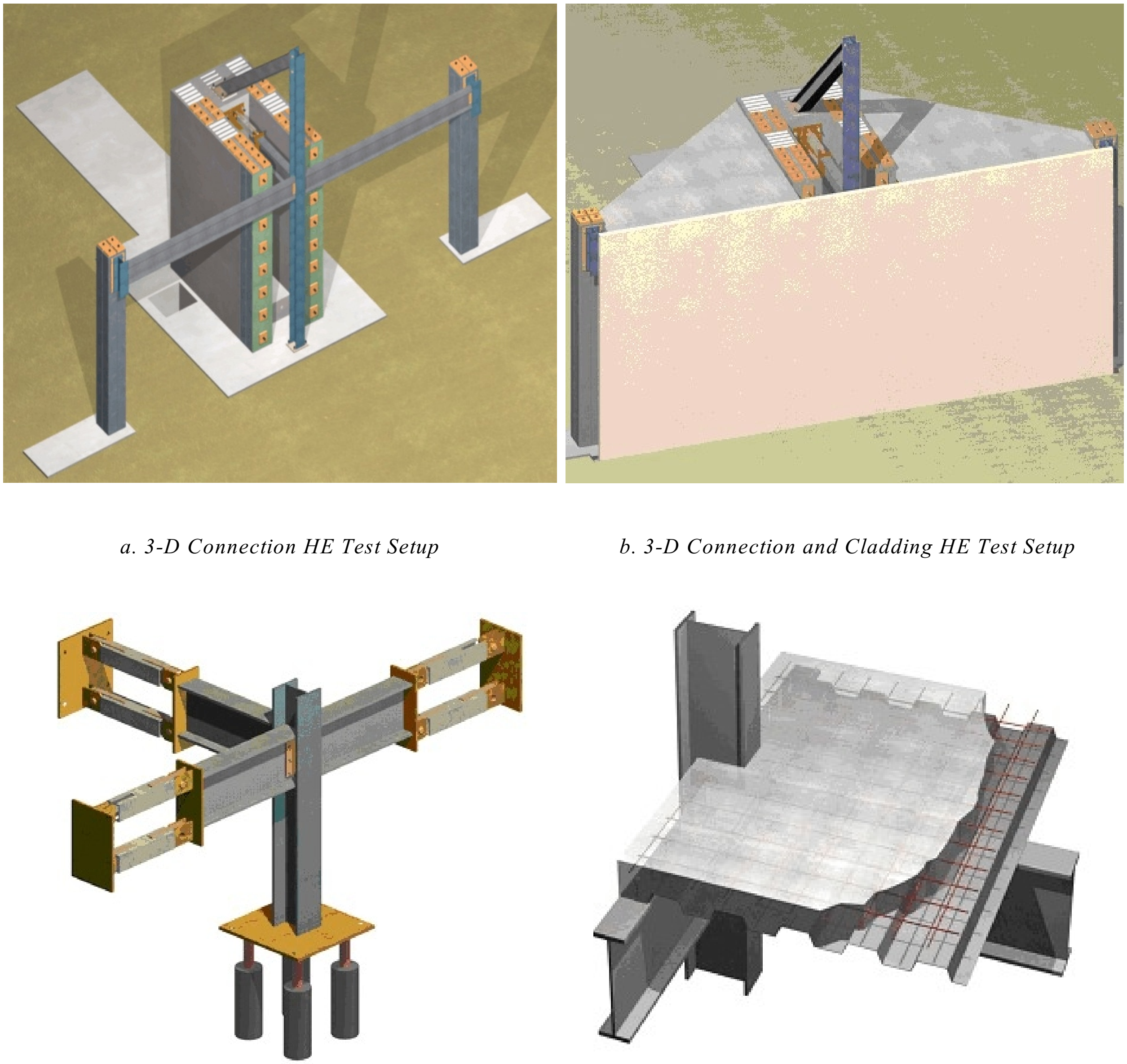
b. Selected two dimensional model

Figure 12 Numerical model for Progressive Collapse Studies (Choi and Krauthammer, 2003)

Preliminary findings from the on going studies on progressive collapse (Choi and Krauthammer 2003) highlight the need to use an external criteria screening (ECS) technique to analyze progressive collapse. Necessary definitions and approaches were developed for material and geometric nonlinearities. A matrix reformation and stiffness reduction technique was described to achieve element elimination effects in the proposed analysis procedure. Matrix partitioning and variable boundary conditions (VBC) techniques were developed to improve solution convergence and stability problem. Such problems might appear in applying a stiffness reduction factor technique that is more advantageous and that leads to relatively shorter computing time. The behaviors and time histories using stress/strain failure criteria for the selected model were compared with those obtained with a general finite element analysis. The behavior of the structural model with a linear material behaved differently than that with a nonlinear material. Structural behaviors with different nonlinear material models were similar. The structural responses obtained with a general finite element procedure were different from those obtained with the modified approach, as presented here. Behavior comparison between those obtained with the developed procedures and those derived with a general procedure is meaningless for the differences of structural system. Buckling was considered as a contributing failure criterion, together with a strain failure criterion. A new solution that can analyze local buckling failure was implemented and inserted as an external module. The collapse started much earlier when buckling was considered than if only a strain criterion was considered. This behavioral difference can indicate that collapse might progress very differently when buckling is considered, and the issues associated with these phenomena require more careful attention.

Besides these on going studies, several proposals are aimed at addressing these issues experimentally in both laboratory impact and field explosive tests. In contrast to static and seismic loading, the loading of steel frames engendered by a terrorist attack will produce some behaviors uniquely related to this type of event. For blast loads, these unique behaviors are primarily due to the rapid load rate—the significant loading is over in less than 5 ms. Loading this rapid often produces direct shear responses that can cause the elements of a section, which are the relatively thin-walled, to shear as if cut by a knife or buckle even before any significant flexure has occurred. High load rates also increase the propensity for brittle cracking and decrease the toughness of welds. In addition, the often highly non-uniform nature of the load can excite twist modes and other higher modes of the section causing instabilities and the general loss of flexural and axial resistance. Besides, the shock flow around complicated geometries (e.g., around the I shape of a column or girder) is not well understood. Therefore, it is difficult to define the anticipated pressure-time histories that would be required for any type of computational effort (e.g., simple design calculations or advanced high-fidelity physics based simulations).

Events like the collapse of the World Trade Center (steel frame) and the Murrah Building (reinforced concrete frame) are challenges to assess and predict because they require the structural system to perform in ways it was never intended. It also requires engineers to understand, quantify, and calculate behaviors that only a few could do even for individual components. When the components (e.g., columns and girders) are combined to form a structural system, its response is far more complicated, and closer to the real conditions. The peculiar nature of steel frames is much more complicated and it includes various interacting modes of response and failure (e.g., local and global buckling, fracture, loss of strength and ductility due to temperature and strain rate effects, etc.). Scaling full size steel building components is much more complicated, as compared to structural concrete systems. This places steel frame multistory buildings in a special class of structure when it comes to predicting their potential for progressive collapse due to a terrorist attack. The uncertainty related to the vulnerability of steel framing of multistory buildings to terrorist attacks is too great to be ignored. Hence, the research program proposed to DTRA (Mosher et al, 2001). Some of these ideas are illustrated in Figures 13 through 15.



a. 3-D Connection HE Test Setup

b. 3-D Connection and Cladding HE Test Setup

c. 3-D Connection Only

d. 3-D Connection with Slab

Figure 13 Some Proposed 3-D Connection Tests (Mosher et al., 2001)

Figure 14 Proposed Full Scale Steel Test Building in the US (Mosher et al, 2001).

Figure 15 Proposed Full Scale Steel Test Building in the UK (Mosher et al., 2001).

SUMMARY

This paper presented an overview of background in blast resistant structural behavior, identified technical difficulties in design and application of such technologies for blast resistant structural steel buildings and systems. This study will be directly linked to other DoD-sponsored R&D that are aimed at enhancing civilian-type buildings' resistance to blast effects, with special attention to preventing progressive collapse. The approach will include using advanced numerical simulations , precision impact tests in the laboratory, and field high explosive tests on components, assemblies, and small- and full-scale buildings. Current and anticipated DoD-sponsored R&D efforts (possibly augmented by other agencies) will be used to define the required test parameters and activities. AISC can facilitate and enable these studies by coordination between the research team, industry, and government agencies.

ACKNOWLEDGMENTS

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Assessing and Minimizing Threats to Buildings

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Abstract

This paper discusses security of buildings due to blast effects. Planning, hazard definitions are presented first. It is shown that there are three integrated phases in how buildings are affected to blast effects: outside the building envelope, the building envelope itself, and inside the building envelope. Each of these phases is discussed. Different mitigating strategies are presented.

1 PLANNING

Planning to reduce blast hazards for buildings involves several steps: the definition of the hazard, H, site considerations, building envelope considerations, structural and non-structural considerations. A successful planning would optimize all of those considerations, so as to minimize the harmful effects on the building and its inhabitants, while minimizing the financial costs. If we assume that the different building and site categories that affect blast hazard are B_i, and that the respective representative costs are C_i. In the planning stage, the relative costs of each mitigating solution should be estimated. The total mitigation cost for a given hazard, H is

$$C_H = \sum_i C_i \ B_i \tag{1}$$

A careful balance of different mitigating solution should result in the minimum cost for the project.

1.1 New Buildings

In new buildings, the site conditions (setback outside the building envelope) represent an easy way to reduce the blast hazard to the building: larger distances quickly reduce harmful blast pressures. This, in turn, result in smaller costs of building mitigating measures. Thus, one of the most important planning issues is: how much setback vs. how much building mitigating measure? In urban area, where setback choices are limited, the emphasis would be on building mitigation measures. In suburban or rural areas, the setbacks are utilized more in securing the building.

In addition to the balance between setbacks and building mitigation measures as a whole, a balance of those building mitigation measure themselves is needed. The building envelope decisions represent a challenging task of balancing. Also, the interior of the building needs to be investigated and the optimum decisions must be made.

1.2 Existing Buildings

Existing buildings represent a far more difficult task. The building site is usually difficult to change. This is particularly true in urban areas, where social and cultural factors limit the introduction of sizable setback, if any. Thus most of the mitigating measures is delegated to the building envelope, and the interiors of the building. In numerous situations, the existing building envelopes and the interior of the buildings were not designed for blast mitigation in mind, resulting in limiting choices for the professionals. A careful application of equation 1 is needed most for existing building. The professionals (engineers and architects) must work closely with building officials and inhabitants to define as many creative, and inexpensive, mitigating solutions as possible. This is done usually by setting a list of all available mitigating methods, and their effectiveness. Then a process of prioritization is performed by all parties. This will ensure that the available funds are spent efficiently.

2 HAZARDS

Figure 1 shows the hazards-Building interaction. The initiation event is the blast event. The main components in the blast event are the distance (standoff, R, ft.) and the weight of the bomb, W, pounds, usually measured in an equivalent TNT weight. The detonation of a bomb with an equivalent TNT weight of W at a distance of R, ft. would cause a blast pressure time history as shown in figure 2. Such a time history can be idealized as a triangular wave form, with maximum amplitude of p, psi, and total duration of t_d, milliseconds. The short duration of the blast wave form have a major implication: it excite many higher modes in the structure, the building envelope and the non-structural components. The very high pressure amplitude p has an equally profound implication: it results, in general, in a nonlinear behavior of all of those systems.

The design values of p and W is governed by several standards. The choice of the applicable standard is usually done by the building owner. Sometime, the design professional and the building occupant participate in making this decision. For more details on the blast pressure properties, the reader is referred to the works of Mays and Smith, 1995.

After the blast pressure arrives to the building, it will affect both the building envelope and the building structural system, see figure 1. The hazards that are generated by the building envelope are the shards and fragmentation of the building envelope components.

The hazards that are generated by the structural system are the potential of progressive collapse; progressive collapse of structures is not discussed in this paper.

Finally, figure 1 shows the next level of hazards; the interior building components (generally referred to as non-structural components). Two categories fall under the interior components: interior architectural systems and mechanical/electrical and plumbing systems (including elevators). In the case of failure of either structural or building envelope, the internal building systems can fail. The hazard from such a failure can be either direct, e.g., interior partition harming occupants, or indirect. An indirect hazard can be a loss of functionality of a life support system, such as an elevator, of electric transformer.

In assessing different mitigating measures for any building, the flow of hazards throughout the building, as represented in figure 1, should be carefully considered. This includes all the hazards sources as well as the interdependence of these hazards. It should be mentioned that the structural progressive failure phenomenon is out of scope of this paper, for further information, the reader might want to refer to the publication by the Research Council on Performance of Structures (1972) or The work that was edited by R.B. Malla (1993).

3 OUTSIDE THE BUILDING ENVELOPE (SITE CONSIDERATIONS)

Site considerations in hazard reductions are affected by such parameters as availability of surrounding land, topography and type of available landscaping. The availability of land is perhaps the most important parameter. In urban areas, no, or limited land is available for use to increase, or control, the setback around the building. In suburban or rural areas, the land is more readily available, so larger setbacks around the building can be utilized in the overall planning stage.

The topography of the site can play a major factor in hazard reduction. Existence, of topographical features such as hills, lakes, rivers, nearby roadway systems can all have positive or negative effects on desired building protection. In general, there are no pre-set rules to govern this issue. It is a case-dependent issue, and must be handled accordingly.

The landscaping of the site around the building can be designed (for new buildings), or changed (for existing buildings) to improve the security of the building. Some desired features would be fountains, pools and moats. Use of thick shrubs and / or trees can help in acting as barriers for vehicles; however, it can act as hiding places.

When landscaping features can’t be used in a particular site, it can be compliments or replaced by a set of barriers. The main purpose of the barriers is to prevent vehicles from coming close to the building. The barriers can have many shape and forms. They can be heavy planters, steel of concrete bollards, etc. In addition to the system of barriers, a system of gates might also be needed for complete protection of the site.

The barriers must be designed to meet specific demands, namely weight and speed of vehicle. There are several standards for barrier designs; the reader is referred to some of those standards for more information. It should be noted that there are situations that the type of soil or the type of construction will result in a case-specific barrier design. An example of such a situation is the existence of retaining wall. To anchor a standard bollard on top of an existing wall, without retrofitting the retaining wall would result in an unconservative/unsafe situation.

There are several standard designs of gates that the professionals can use to compliment the design of a secure site. Some gates are manually operated, while others are automatic. The designers should coordinate the appropriate choice of the gate type and location with the building owners and occupants.

4 BUILDING ENVELOPE

Building envelope is one of the exposed systems in the building (the other is the structural systems). The building envelope, as shown in figure 1, is affected directly by blast pressures. If not designed properly, it can harm either the occupants, or the other major components, inside the building. During the Oklahoma City Bombing, numerous human losses were encountered because of the failures of different building envelopes. See the report of ASCE, 1996 for further details.

Building envelope can be constructed of metals, cement-based materials, and/or glass. Curtainwalls and cladding can be part, or whole of the building envelope. For secure building envelope, it should be designed to transmit the postulated blast pressures safely into the supporting structural system.

In designing building envelopes, it should be remembered that different hazards affect building envelope; including seismic, wind, as well as blast hazards. There are some similarities between seismic and blast hazards such as both hazards tend to exert dynamic forces on the system, thus causing it to dislodge from its anchors. Mitigation methods for the harmful effects of earthquake hazards are well documented. The common seismic design for building envelope components is based on estimating a seismic design force, $F_p = a_p\ m_p$, where m_p is the mass of the system under consideration and a_p is an adequate acceleration value. The system under consideration is then anchored to the main structure so that the anchoring mechanism can resist the force F_p. In addition, qualitative seismic detailing might be needed to ensure that the building envelope components do not sway or fall during an earthquake.

Applying the above methodology to blast condition is problematic. First, the design seismic force, F_p, is based on an assumed seismic acceleration, a_p. Effective blast force on a particular nonstructural system depends on the expected blast pressure that will affect the system. Second, the qualitative seismic detailing for anchoring building envelope components to the building are all base on expected seismic deformation modes. Blast deformation modes can be different from the seismic ones; in many cases they are the opposite. Blast-specific approach to the design and detailing of building envelope components to mitigate blast conditions is needed.

There are several important considerations that are needed in a blast-specific design for building envelope:

Considerations of dynamic effects: As was discussed above, the short duration of blast events would excite high frequency modes. Since many of the components of building envelope have frequencies that are within the blast pressure range, their dynamic responses should be accounted for in the design.

Rate effects on Strength and ultimate strains: IT is well known that high loading rates increases the strength of materials. However, these same high loading rates decreases the ultimate strains of materials. This phenomenon should be included in the design.

Balanced Design: Since the building envelope is usually constructed from several components that are made of different materials, it is important to ensure that the load path throughout all the components and the jointing between these components is strong and ductile enough for the postulated blast load. Any weak link can render the whole building assembly unsafe.

Glazing: Shattering of glass is a particularly worrisome occurrence during a blast event. During different terrorist acts, the shattering of glass was the major cause of human fatalities and injuries. The lack of ductile behavior and the large coefficient of variation in design parameter make it essential that glass design be addressed properly.

Anchoring to Structural systems: Good design of building envelope is not complete without appropriate anchoring to the supporting structural systems. If there is any potential of interaction between the building envelope and the supporting structural system, it should be accounted for in the design.

Un-Reinforced masonry (URM) bearing walls: In many building the URM is used extensively in the building envelope. URM is not ductile systems, and is fairly weak in the transverse direction. Some retrofit measures should be employed for existing buildings that have URMs. For new construction, some form of reinforcement should be used in exterior masonry walls.

A safe and secure building envelope will result from following the above rules. For more information about curtainwall design and behavior, refer to the work by Zhou, 2002.

5 INSIDE THE ENVELOPE

Figure 1 shows the two categories of systems inside the building envelope. They are the Architectural Systems (AS), and the Mechanical, Electrical and Plumping (MEP) systems. The architectural systems include, but not limited to interior partitions and walls, ceilings, light fixtures, furniture and elevated floors, if any. MEPs include generators, transformers, elevators, chillers, etc.

The safe design for blast effects include appropriate anchoring, hardened enclosures and simply installing important MEPs always from harm's way. For example, it is prudent to locate generators, and other essential equipments, deep inside the building, rather than near the outside. Another subtle, but important consideration is the possible harmful effects that seismic considerations might have on blast considerations. For example, whenever a seismic restraint (snubber) or seismic (or noise) isolation system is placed with a heavy machinery (or even a light weight clean room) the effectiveness of any of those measures should be considered during blast event. In particular, the differences in the operational frequency ranges for different hazards must be studies, see figure 3.

6 CONCLUSIONS

The different aspects of safety of buildings during a blast event were discussed above (not including the structural systems). The flow of hazards from the source all the way to the inside the buildings were illustrated. The change of the type of hazard, from blast pressures to flying glass shards to the disabling of important equipments was highlighted. In addition, different mitigation options for each of the stages were mentioned.

Secure and safe buildings require early planning as well as integral mitigating strategy, as highlighted by equation1. Without such an integral approach, the result will not be cost effective. Worst yet, it may not end up in a safe building.

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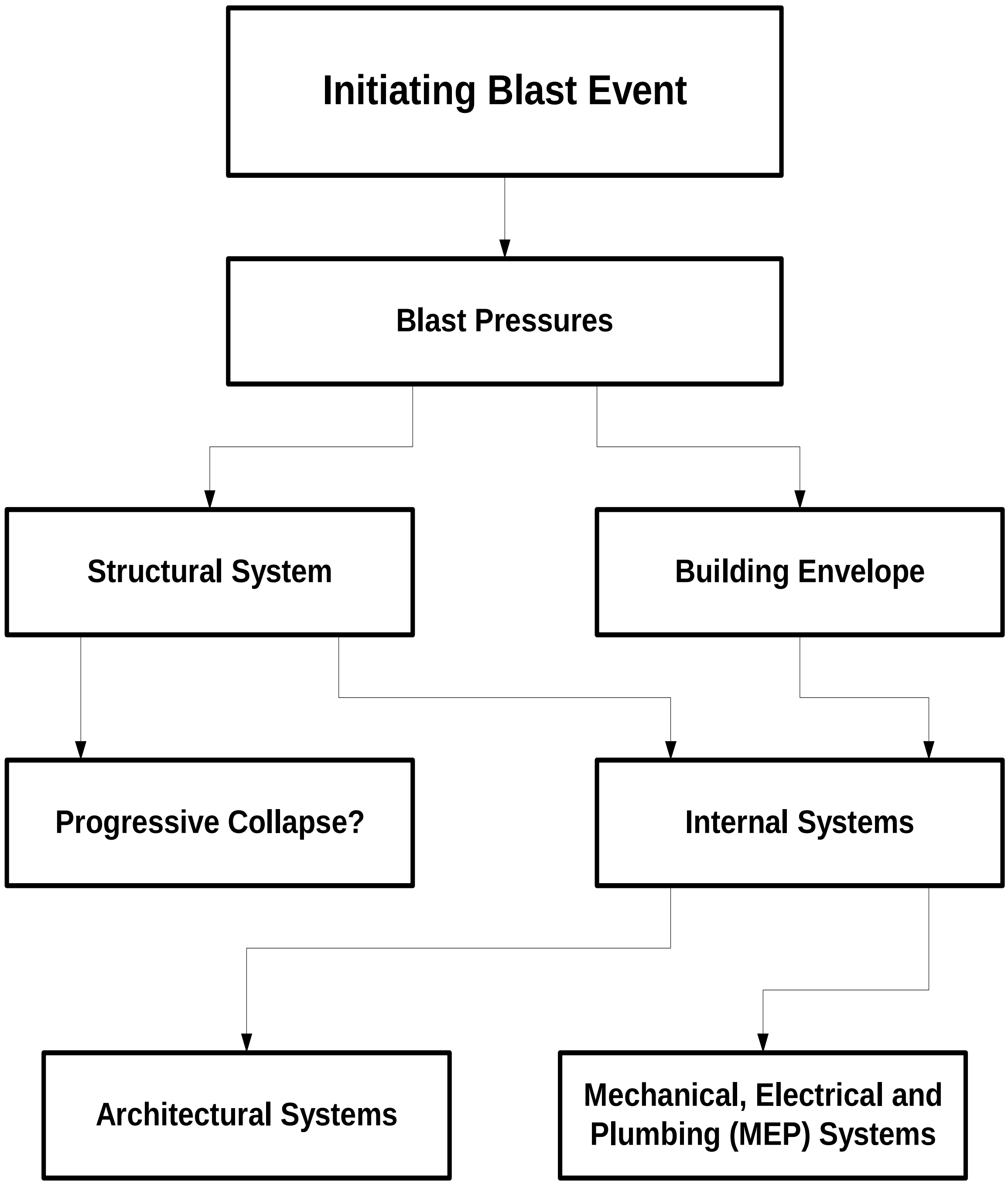

Figure 1 – Different Hazards Resulting From a Blast Event

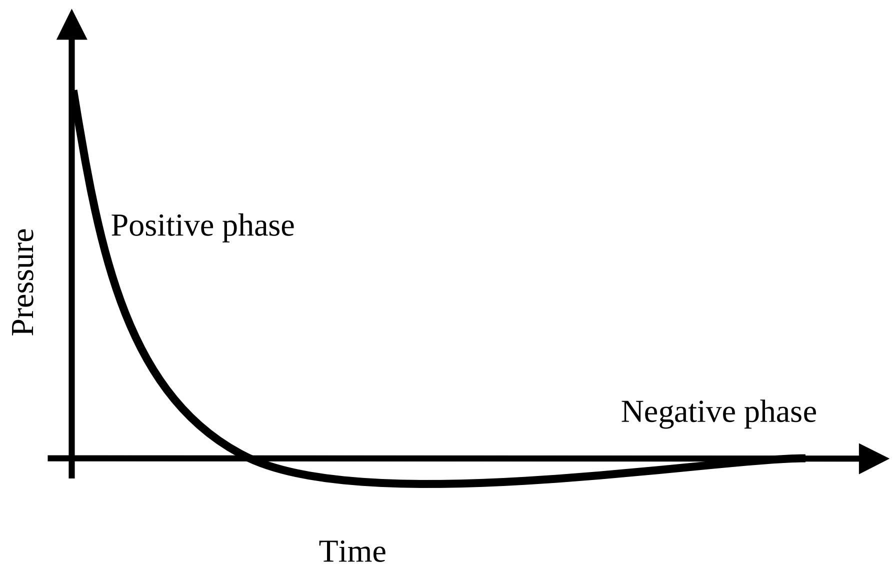

Figure 2 – Typical Blast Pressure Wave Form

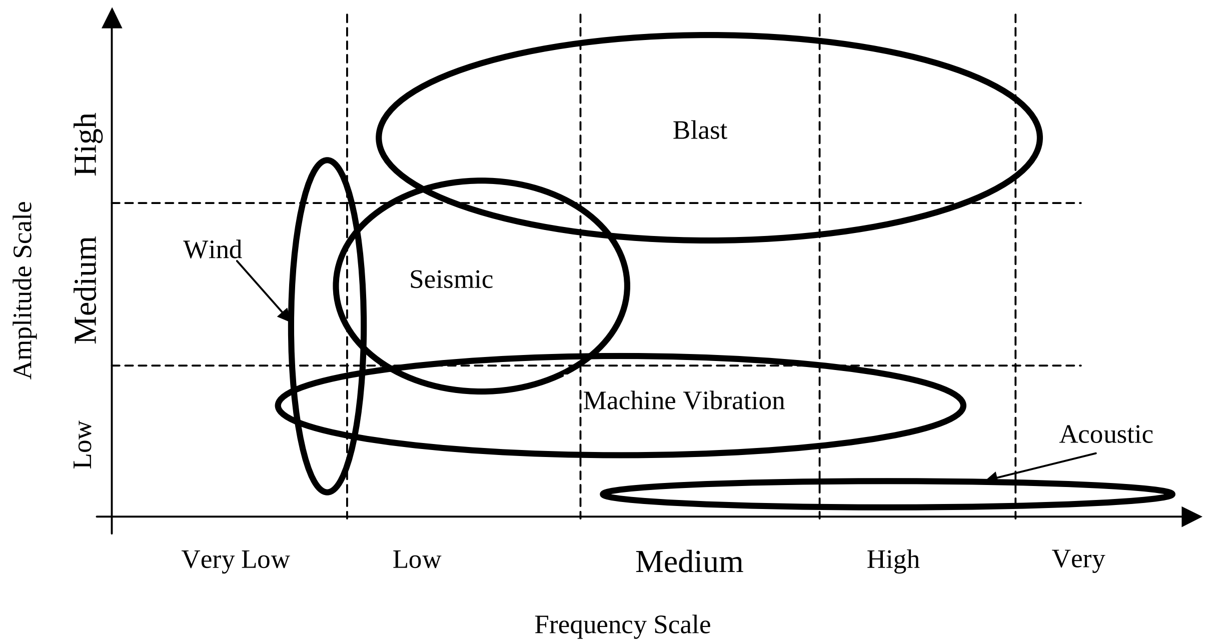

Figure 3 – Qualitative Frequency-Amplitude distribution for different Hazards

SIMPLE NONLINEAR STATIC ANALYSIS PROCEDURE FOR PROGRESSIVE COLLAPSE EVALUATION

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Abstract: There is a concern about progressive collapse of buildings. A simple structural design criterion including definitions for key or important members in a structure is proposed and a single-degree-freedom model is created first to illustrate the analysis procedure of progressive collapse. Then, a nonlinear static analysis procedure for existing buildings is presented. Evaluation of a six-story concrete structure is carried out based on this procedure and the result of this simplified approach is compared with the calculation from a nonlinear dynamic procedure.

Key Words: Progressive Collapse, Criterion, Nonlinear, Static

INTRODUCTION

After the Murrah Building collapse in Oklahoma City, there is an interest in progressive collapse potential evaluation. The General Services Administration issued "Progressive Collapse Analysis and Design Guidelines – for New Federal Office Buildings and Major Modernization Projects" (GSA, 2003) as a guideline for engineers involved in the evaluations. The guidelines and additional information including standards, references, test data, computer programs and reference projects can be found at the GSA website http://www.oca.gsa.gov/. More information can also be found in other codes including the New York City Code Chapter 18 of Rules and Regulations, The British code and the ACI code. One requirement common to various codes is to verify the presence of an alternate load path in case a column is removed and some prescriptive requirements are not met. Another is to strengthen the key or important members defined as those whose failure can create extensive damage to the structure.

The goal of a designer is to produce designs that are reliable and to avoid creating structures that can have major collapses due to damage in small areas or failure of single elements. To realize these designs engineers should have an overall concept for the structural design, should determine what members are important or key to their design, and have the tools to assess the extent of structural damage if a structural member fails.

The designer should be aware that in an optimally designed structure every square foot has the same reliability for every accidental loading case considered. If this is not the case, one part of the structure is stronger than another and the designer should reinforce the area more prone to fail with material from the area less prone to fail. In practice is most probably impossible to obtain the same reliability everywhere hence the goal should be to obtain a minimum allowable reliability for every square foot of structure.

The designer can identify the key or important members in a structure, as the ones that have bigger influence area and/or carry more loads and/or have higher strain energy and/or those whose failure can create extensive collapse. The bigger the load a member carries or the bigger the influence area the more load the structure has to redistribute or the bigger is the extent of the damage in case the member fails. The strain energy under dead loads is a measure of the work of the member; the more work the member does the more significant is the member. As the strain energy is the combination of the load and the section properties this is a more complex indicator than the total load or the influence area that are only function of one parameter. Other important indicator is the reserve capacity of the member defined as the ratio of the energy the member can absorb until it fails to the strain energy it has under dead loads. Another indicator is the ratio of the strain energy of the member to the volume of the member or strain energy density that gives an idea of what members are more stressed out. In an ideal structure this density is uniform throughout the structure. The strain energy density is a more exact indicator than the stress level because it takes into consideration the whole volume of the member not just a section. Finally the failure of secondary members that brace or stabilize the primary members can produce significant damage to the structure. These members have a small influence area, carry small loads and have small strain energy. These members cannot be easily associated to a numerical parameter. The designer will identify these members also as key or important to the redundancy of the structure.

The rest of this paper explains to the designer some fundamental concepts to compute the structural behavior under a column removal scenario.

This paper is a further exploration of the energy balance method used by Graham H. Powell in "Collapse Analysis Made Easy (More or Less)" (Graham H. Powell, 2003). The method is the technical basis for the RAM Perform-Collapse computer program

In this paper, a single-degree-freedom nonlinear system, consisting of a nonlinear spring and a concentrate mass, is created first to illustrate the procedure of progressive collapse. Section I is the detailed description. Section II presents a nonlinear static analysis procedure for existing buildings. The basic concept of the procedure is energy balance, i.e., the structure must absorb the potential energy generated due to the removal of one column. Section III is an example to illustrate the above procedure. Evaluation of a six-story concrete structure is carried out and the result is compared with a nonlinear dynamic procedure

I. ILLUSTRATION OF PROGRESSIVE COLLAPSE

The progressive collapse procedure is similar to a single-degree-freedom system as shown in Figure 1. Figure 2 is the property of the nonlinear spring. Point A, B, C, D and E in Figure 1 and Figure 2 denote same state. Table 1 is the list of system variables.

Energy dissipated in the structure due to damping is minimum compared with the energy absorbed due to plastic deformation. Thus, damping is not considered in the following description of the progressive collapse procedure.

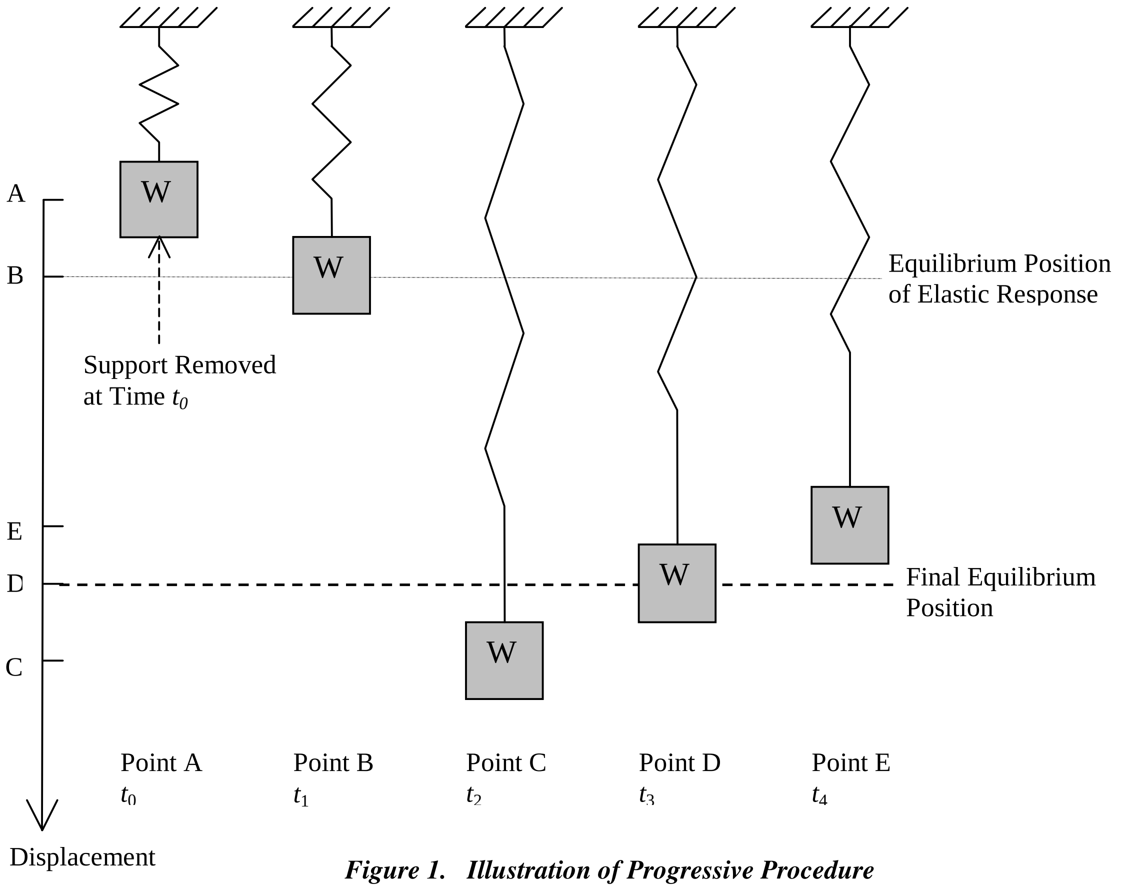

Figure 1. Illustration of Progressive Procedure

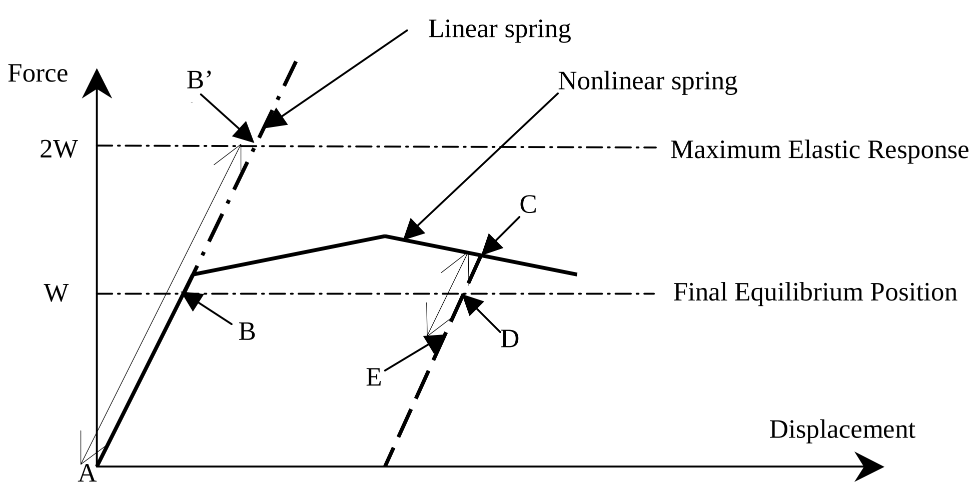

Figure 2. Force vs. Displacement Diagram of Spring

Table 1 System Variable

Point	Force	Potential Energy	Kinetic Energy	Energy absorbed by Spring
A	Down	-W*A[a]	0	0
B	Zero	-W*B[a]	+	+
C	Up	-W*C[a]	0	W*C[a]
D	Zero	-W*D[a]	+	+
E	Down	-W*E[a]	0	W*E[a]

[a]: A, B, C, D, and E denote the displacement coordinate at those points.

- At point A, when the column is removed, the system has the maximum potential energy. Since the force in the spring is zero at this time, the system is falling down due to the weight of the system, W.

- From point A to B, the downward velocity increases and reaches its maximum at point B. After point B, the downward velocity decreases because the force in the spring is greater than the weight of the system, W. If the yield capacity is greater than 2W, the response of the system is linear static as the straight line AB' shown in Figure 2.

- At point C, the falling system has zero velocity and all the potential energy is absorbed by the spring. Point C can be obtained by above energy balance condition. After point C, the system starts rebound because force in the spring is greater than the weight of the system, W.

- At point D, the system has maximum upward velocity. From point D to point E, the upward velocity decreases and becomes zero at point E. If the unloading curve of the spring is straight, it can be seen that distance CD equal to DE.

- Point D will be the final state.

Several conclusions can be reached:

- For the system not to fail, the strength of the spring at point C must be greater than the weight of the system.

- If the weight of the system is greater than the maximum strength capacity of the spring, the system will fail.

- If the weight of the system is smaller that half of the yield strength of the spring, the system has only elastic response and will not collapse.

- The magnitude of the vibration between point C and point E is generally small compared with the elastic response and generally there is no load reversal. Hence the system will not fail as it oscillates around point D.

II. NONLINEAR STATIC ANALYSIS PROCEDURE

Following is a description of the proposed nonlinear static analysis procedure:

1. Put a load proportional to the reaction of the removed column and increase it gradually to get the pushover curve of the structure.

2. If the reaction is less than half of the yield strength of the pushover curve, the structure has low potential for progressive collapse.

3. If the reaction is greater than the maximum strength of the pushover curve, the structure has high potential for progressive collapse.

4. If conditions of 2 and 3 are not satisfied, generate the capacity curve and compare it with the load curve. This step is illustrated in Section III.

The above procedure can be used as a preliminary screen procedure to verify if conditions of step 2 or 3 are satisfied.

Section III is an example to illustrate step 4. The basic concept is energy balance, i.e., the structure must absorb the potential energy generated due to the removal of one column. The capacity curve is generated by dividing the energy absorbed by the structure, area below the pushover curve, by the displacement. The capacity curve is then compared with the load curve, which is a straight line parallel to X axis with the magnitude equal to the weight supported by the removed column.

III. ANALYSIS OF A SIX-STORY CONCRETE BUILDING

A six-story concrete building is analyzed to illustrate the proposed nonlinear static analysis procedure.

Only one 2-D elevation is considered in this report for simplicity. Tables 2 and 3 are the properties of beams and columns and Figure 3 is the structure elevation:

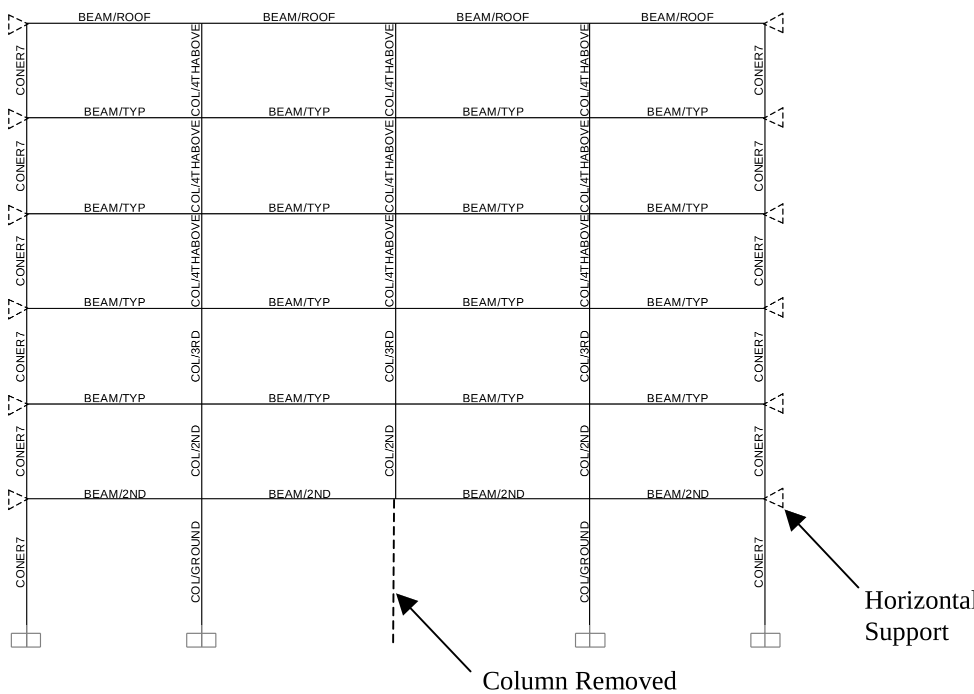

Figure 3. Elevation of the Structure

Table 2 Six-Story Building Beam Reinforcement and Capacity

Floor	Beam Size Depth (in)	Beam Size Width (in)	Flange Width (in)	Flange Add. Top Rebar (in^2)	Top Bar (in^2)	Bottom Bar (in^2)	Stirrup (in^2)	Stirrup Space	Shear Cap kip	Positive Mom. (k-f)	Negative Mom. (k-f)
RF	39	14	72	1.76	1.2	1.58	0.22	12	118	359	642
6	29.5	14	32	1.24	1.2	1.58	0.22	12	87	262	389
5	29.5	14	32	1.24	1.2	1.58	0.22	12	87	262	389
4	29.5	14	32	1.24	1.2	1.58	0.22	12	87	262	389
3	29.5	14	32	1.24	1.2	1.58	0.22	12	87	262	389
2	49.5	14	52	1.86	1.2	2.54	0.22	12	152	739	863

Table 3 Existing Column Size and Reinforcement

Floor	Six-Story Building			
	Middle Col		Corner Col	
	Size (in)	Rebar	Size	Rebar
6	40X16	6#7	L shape with each leg 40X16	12#7
5		6#7		12#7
4		6#8		12#7
3		6#8		12#7
2		6#9		12#7
1		6#10		12#7

Plastic moment hinges and axial hinges are assigned to beam ends. Moment hinge properties are taken from FEMA 356 (FEMA, 2000) as shown in Figure 4. Figure 5 is the axial hinge property diagram, assuming infinite deformation capacity. P_y for the beams is calculated taking only into consideration the rebar located in the area that is in compression due to flexure. We assume that the rebar located in the area that is in tension due to flexure has yielded. Columns are assumed to remain elastic due to their size. The hinge modeling does not account for interaction between the axial force and the bending moment. Large displacement analysis is used to engage cable action. Horizontal supports are provided to simulate restraint actions of the other bents. The actual solution is bracketed between having horizontal supports that enhance cable action and decrease column bending and having no horizontal supports that neglects any restraint provided by the floor slab.

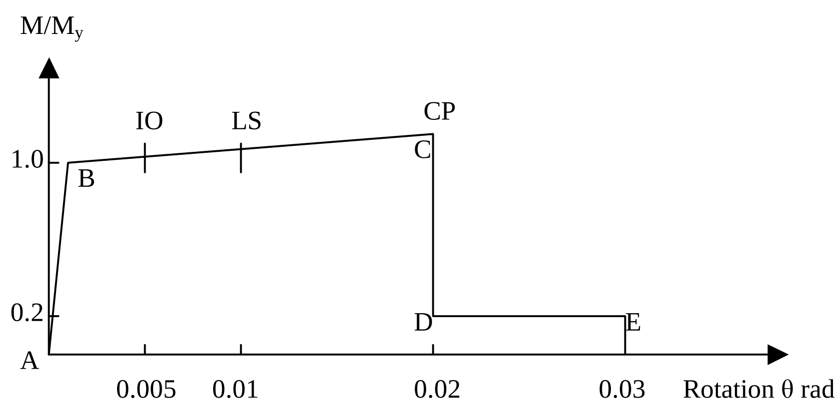

Figure 4. Moment Hinge Properties

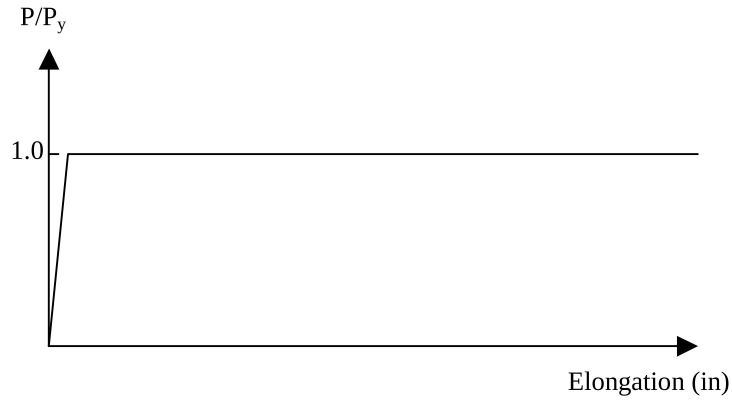

Figure 5. Axial Hinge Properties

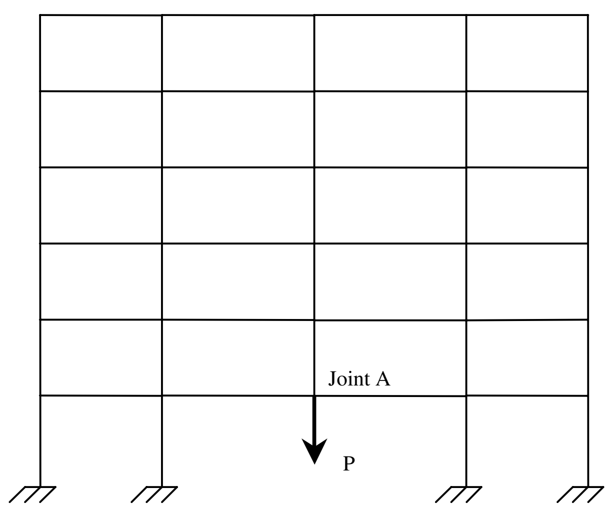

Figure 6. Loading for Pushover Analysis Procedure

Figure 6 shows the loading condition to get the pushover curve. For simplicity, the structure is set up with no gravity load and a missing column. A more accurate solution is to include all the gravity loads present at the time of the column removal. The load P is equal to the reaction of the column removed, 440 kips in this case. For this example, we apply a maximum displacement of 144”. The displacement control analysis computes at each displacement step the amount of load required to create the displacement.

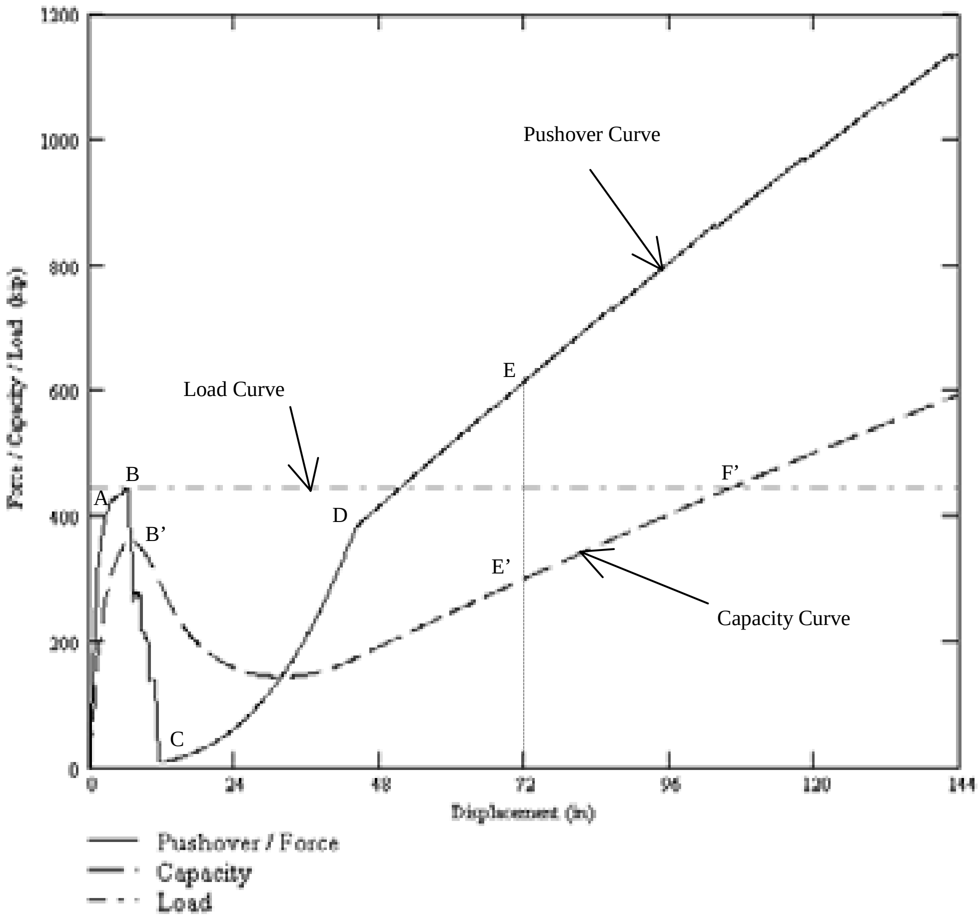

Figure 7. Pushover Curve, Capacity Curve, and Load Curve

Figure 7 is the pushover curve. Point A, B, C, D, and E on the pushover curve indicates different stages of structure behavior. Before point A, the structure behaves elastically with point A corresponding to the yielding of the structure. After yielding, the beams strength hardened from point A to B. At point B, the hinges fail and there is an abrupt drop. Curve CD indicates that the structure begins to pick up load due to cable action. At point D, reinforcement bars yield due to tension and the slope of the pushover curve becomes smaller. Since the model assumes the rebar has infinite deformation capacity, the structure can continue to sustain load without failure.

The area below the pushover curve is the energy that the structure can absorb. If we divide the energy below the pushover curve by the corresponding displacement, we can get the capacity curve of the structure. For example, point E' on the capacity curve is obtained by dividing area below OABCDE by the displacement at E, 72" in this case. The pushover curve and capacity curve are characteristics of the structure under given load condition.

The load curve is straight in this case, which is equal to the reaction of the removed column, 440 kips in this case. From Figure 7, it can be seen that the capacity curve is lower than the load curve before point F', which means that the structure can not absorb the potential energy before reaching the displacement corresponding to point F'. It is obvious that the structure will collapse if it deflects as much as point F', even if the energy can be balanced at point F. Thus, the conclusion is that the 2-D frame has a high potential for progressive collapse.

Figure 7 also shows that the capacity of the structure is about 80% of the required capacity near point B'.

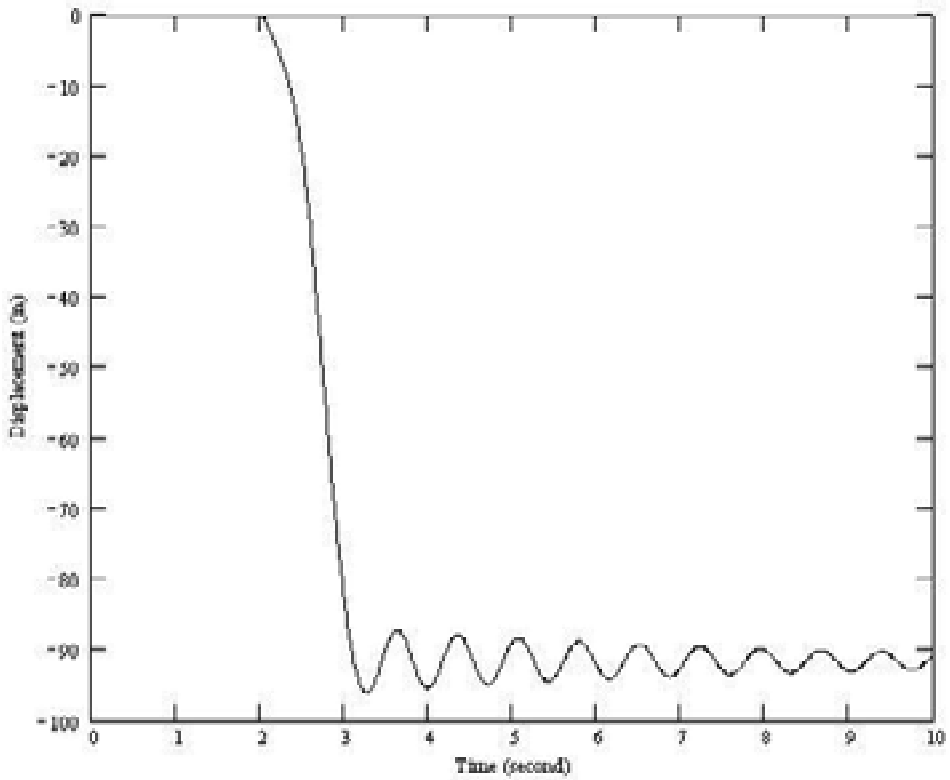

Figure 8. Vertical Displacements vs. Time Diagram

A nonlinear dynamic analysis is also carried out to verify the result of the nonlinear static procedure above. 3% of critical damping is introduced and force equal to the reaction of the column is put on the structure. Figure 8 is the joint vertical displacement response, where the column is removed.

The structure will not collapse because the axial hinge has infinite deformation capacity. The structure will reach its equilibrium at about 93 inches, which is close to 100 inches from the previous nonlinear static analysis. If the load on the structure is reduced to about 80%, the moment hinges will not fail, which is also very close to the nonlinear static analysis.

IV. CONCLUSION

The following conclusions can be reached:

- A simple structural design criterion including definitions for key or important members in a structure is proposed.

- A simple quantitative nonlinear static procedure is proposed for analyzing the progressive collapse potential caused by the removal of a column.

- The proposed nonlinear static procedure gives reasonable results for the example shown. The procedure also gives a quantitative measurement of the potential for progressive collapse.

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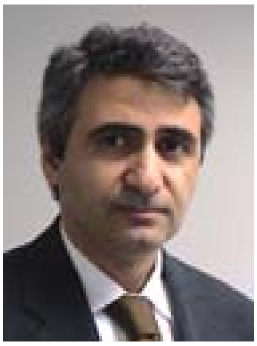
NON-LINEAR STRUCTURAL INTEGRITY ANALYSIS



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ABSTRACT: The provision of British Standard BS5950 is discussed. The intent of the current standard on structural integrity provision is to localize the damage as a result of removal of one member (i.e.: column). A new method for enhancement of the structural integrity requirement is presented here. This method allows localizing the damage in an event of multiple column removal and therefore eliminating the likelihood of disproportionate or progressive collapse. The method utilizes the interaction between beam and slab elements in a three dimensional space by considering the membrane forces generated into the diaphragm by the geometric action of the deformed structure. A series of three dimensional non-linear finite element analyses were performed to simulate the behaviour of the floor system in absence of supporting columns.

NON-LINEAR STRUCTURAL INTEGRITY ANALYSIS

AHMAD RAHIMIAN, PhD, PE, SE   KAMRAN MOAZAMI, PE,

INTRODUCTION

The aim of the disproportionate collapse criteria of the UK Building Regulations and material codes of practice is to ensure that buildings are generally robust and that a local incident does not cause large-scale collapse.

This paper presents a method for enhancing the structural integrity of high rise buildings beyond current practice. This approach focuses on redundancy and enhancement of the alternate load paths. This approach requires three dimensional analysis of the floor framing considering geometric and material nonlinearity.

This approach was developed for enhancing the structural integrity of a high-rise building in the United Kingdom beyond the British Standard. The criterion was set to be that the overall integrity of the structure should not depend on the integrity of any one or two columns.

The design process was developed using three dimensional finite element nonlinear large deformation pushover analyses for various column removal scenarios.

CURRENT REQUIREMENTS OF BRITISH STANDARD

Current British Standard (1) has a descriptive integrity requirement which implies that the descriptive requirement, if met, can accommodate the removal of any one column without initiating a progressive collapse. The descriptive provisions of British code of practice include requirements for internal ties, peripheral (edge) ties and vertical (column) ties with specified capacities. The descriptive criteria, while adequate, will not stand the scrutiny of a conventional analysis which is limited in its prediction due to inherent simplified assumptions on structural and material behaviours. However, recent tests carried out at the University of Berkeley, California, demonstrated that such structures can perform successfully (2)**.**

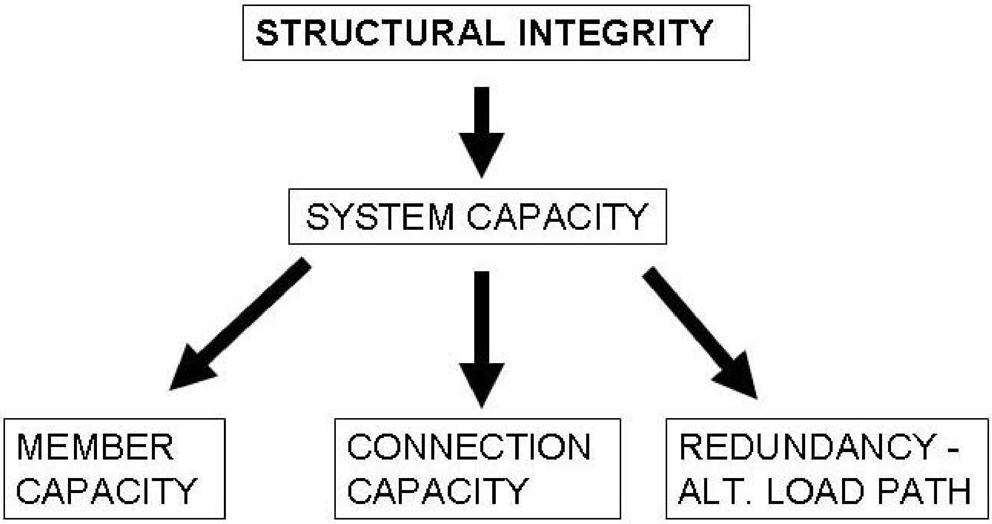
The few standards that address the issue have in general descriptive provisions instead of performance provisions. The descriptive provisions try to enhance the structural integrity without addressing any specific threat or structural performance. The performance provisions, while computationally more demanding, have the advantage of addressing directly the structure's behaviour under a given scenario.

ENHANCED STRUCTURAL INTEGRITY CRITERIA

For this specific project, the current British structural integrity criteria were enhanced to tolerate removal of any two columns within the building.

Generally, any measures with respect to structural integrity aim to enhance the system capacity. The system capacity enhancement is achieved either by enhancing member capacities, ductility or introducing alternate load paths or redundancy.

In this specific project, the goal was to enhance the structure redundancy by expanding on the alternate load path capacity. In order to ensure that the alternate load paths have adequate capacity to transfer the loads, member strength and ductility requirements were reviewed and upgraded.



The two column removal in essence is similar to the single column removal except imposing a higher demand on the remaining structure. In order to activate the secondary load paths the structure will go under a large deformation to the extent that is necessary to engage the self-equilibrating catenary behaviour of the floor system. This obviously requires that all other modes of failures have a higher load carrying capacity.

Figures 1 and 2 show the two-dimensional catenary concepts for single and double column removals. In two-dimensional analysis tie forces at the end of the catenary are required to achieve equilibrium. In general, the horizontal component of the tie force cannot be handled by any reasonable sized columns.

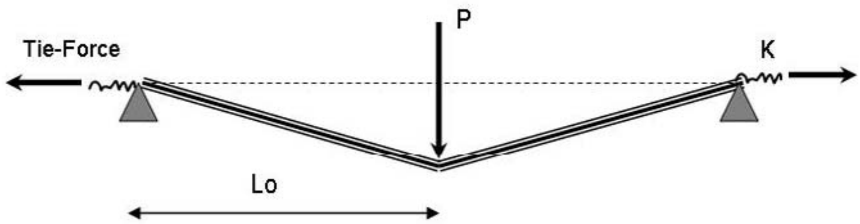


Figure-1: Catenary equilibrium for single column removal

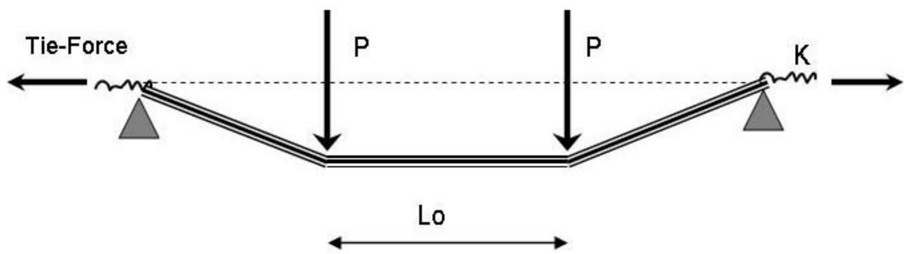


Figure-2: Catenary equilibrium for double column removal

Figure 3 shows the relationship between the two secondary load paths, i.e. beam action and catenary action as a function of beam rotation. The M/MP line shows the beam flexural action and the T/Ty shows the catenary actions. Initially the system under low level of loads acts as pure bending element and as the load and the rotation increases the system will convert itself from a bending element to a catenary element.

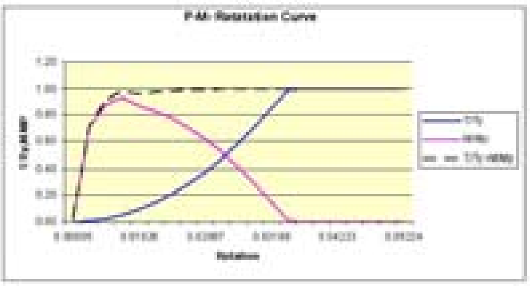


Figure-3: Interaction between "Beam Action" and" Catenary Action"

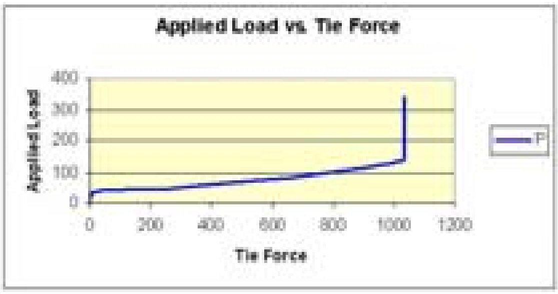


Figure-4: Relationship between applied load and catenary tie force

Figure 4 shows the relationship between vertical load and horizontal tie force for single column removal scenario.

In a two dimensional catenary system the system integrity depends on the capacity of the support to resist the horizontal component of the force. Generally, columns are not designed to receive lateral loads of the magnitude required for the equilibrium of a catenary system.

The advantage of a three dimensional behaviour is that the equilibrating forces are internal to the system. In principle the floor system goes through a deformation that reshapes the floor plate into an inverted dome or a dish, see Figure-5. As a result, the equilibrium of the system does not depend on the capacity of the catenary tie forces at the support. In other words, the radial tensile forces created as a result of the dishing action is balanced with compressive hoop stresses of the composite floor system.

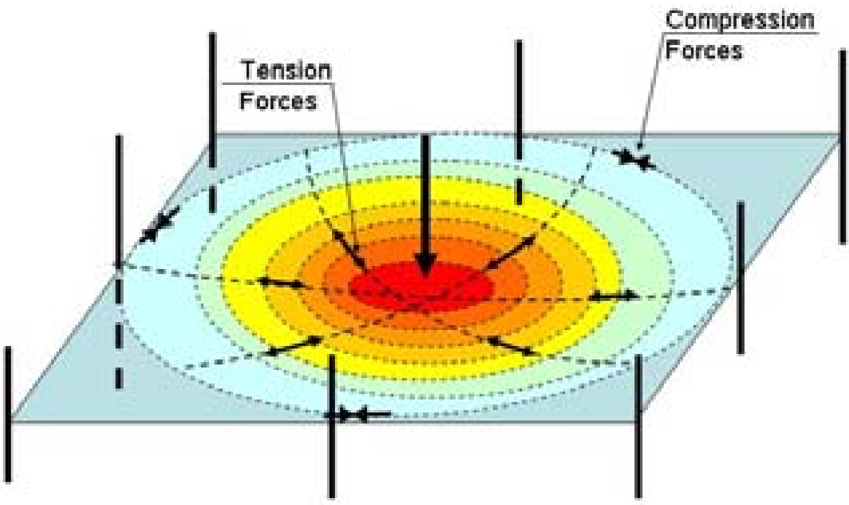


Figure-5: Three dimensional catenary shell action of the composite floor a system.

CASE STUDY

The building is 35-stories 165 meters tall. The building lateral system is comprised of three independent concrete core wall systems surrounding the stairs, elevators and mechanical zones. The floor framing system is steel construction with composite metal deck and concrete slab supported by steel columns and concrete walls, see fig.6. Average column spacing is 9 meters.

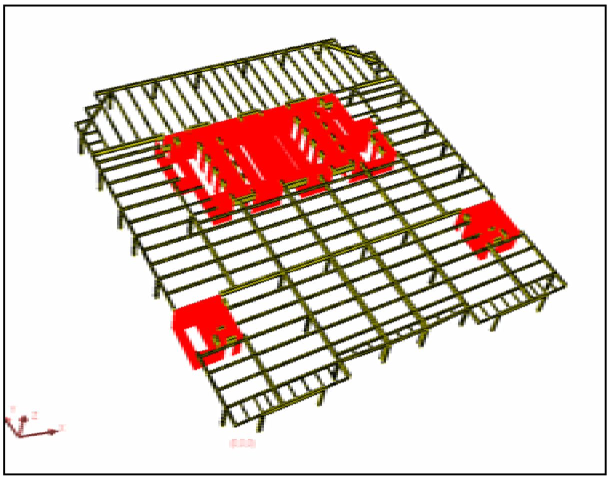


Figure-6: typical floor framing

GEOMETRIC AND MATERIAL NON-LINEAR FINITE ELEMENT ANALYSIS

To study the behaviour of the structure, a three-dimensional model was created using the "LARSA Integrated Linear and Non-linear Finite Element Analysis and Design" computer program. The beam element includes the effects of both geometric and material non-linearity. The analysis involves continual iterations of the stiffness matrix at points along the length of the beam as the load is gradually applied. This iteration process continues after the beam has yielded and redistributes the stresses to the adjacent structure as the load is increased. This analysis involves developing appropriate demand and capacity curves that are utilised to assess - and ultimately prevent - structural collapse.

A three-dimensional model of a whole typical structural floor was created as shown in Figure-6. This structural model was then analysed for a series of column removal scenarios. It was considered that when a column was removed the line of columns above would act as a hanger so that every floor structure would be forced to behave in a similar fashion to the floor being analysed; the hanger columns would effectively become redundant.

The concrete floor slab was modelled as thin shell elements connected to the steel floor beams. The tensile membrane stress in the concrete slab was monitored, and cracking of the concrete slab was considered. The cracking of the concrete in the principal tensile direction reduces the stiffness of the shell in the radial direction. This mechanism, due to strain compatibility requirement, sheds the tensile radial forces to slab reinforcement as well as the steel framing grid acting in the radial direction.

LIMITING CRITERIA

It was necessary to establish criteria in order to confirm the structural integrity after the columns are removed. The first criterion was to ensure that the columns adjoining the removed columns were not overloaded. This is achieved by performing strength check on the remaining structure, especially the adjacent columns using a realistic extreme event service load.

Alternatively this can be carried out by checking manually that columns adjacent to the removed columns can support the new load due to enlarged tributary area.

In extreme events, maximum deflection is not directly a limiting criterion. The behaviour can be considered acceptable if the strain in the yielded beam is limited to prevent collapse. Consequently, a maximum strain limit of 5% was adopted rather than imposing maximum deflection criteria.

The distribution of forces to the adjoining structure was also monitored. The results show that the bending moment diagram, together with the catenary axial force, reflects the element yield diagram by showing that the–bending–moment beyond the beam's plastic capacity as being yielded. The beam axial forces also display the effect of catenary action by activating the surrounding grillage of beams and the concrete slab.

The floor plate stresses were closely scrutinised to prevent failure. The forces were distributed between the catenary action of the steel structure and membrane action of the slab. In the extreme event the concrete floor will act as a thin shell developing radial and hoop stresses. Concrete floor on metal deck can easily accommodate the compressive hoop stresses and its reinforcement was primarily upgraded to carry the tensile stresses. The tensile capacity of the concrete as well as metal deck is ignored.

LOADING CASE AND SCENARIOS

The pushover nonlinear analysis was performed under full dead load of the structure and superimposed dead load. 50% of live load was considered.

Using the above model, various column removal scenarios were analysed. Obviously, in each scenario a different load path is activated.

ANALYSIS RESULTS

While in each scenario a different load path is activated, the performances are similar and can be categorized in three groups i.e.: Interior column removal, Perimeter column removal, Corner column removal.

Interior Column Removal:

Figures-7 and 8 show the plate principal compressive and tensile stresses. Figure-9 shows plastic hinge formation in the main catenary framing member. Figure-10 shows the catenary forces in various framing members.

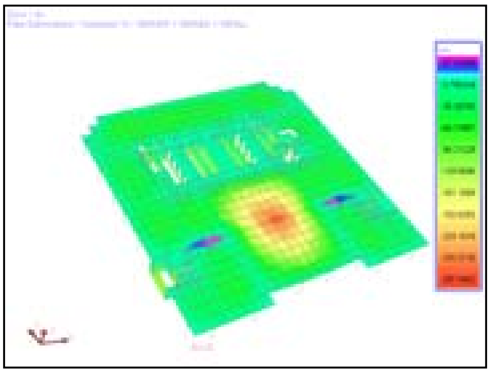


Figure-7: Principal compressive hoop stresses in the concrete floor plate

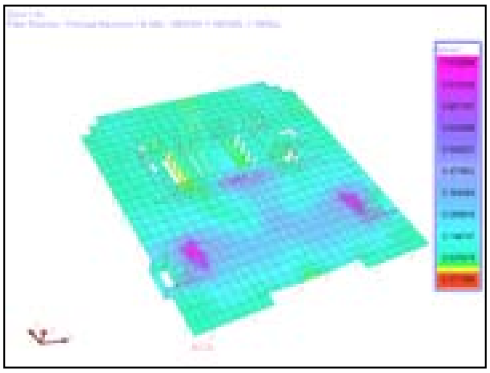


Figure-8: Principal tensile radial stresses in the concrete floor plate

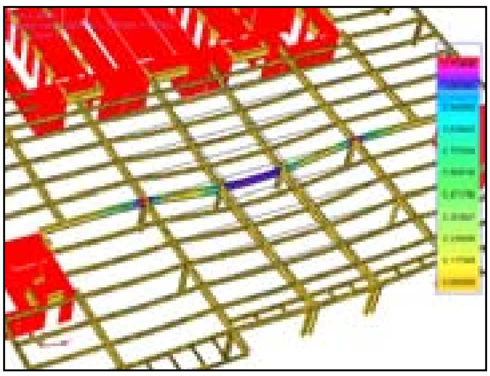


Figure-9: Plastic hinge formation in the main catenary framing element

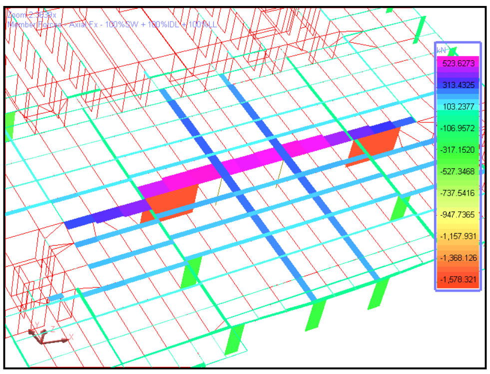


Figure-10: Catenary force diagram

Exterior Column Removal:

Figures- 11 & 12 show the plate principal compressive and tensile stresses. Obviously the exterior condition does not allow a complete formation of catenary shell action. As a result additional stress is imposed on the spandrel beams. Figure-13 shows plastic hinge formation in the main catenary framing member. Figure-14 shows the catenary forces in various framing members.

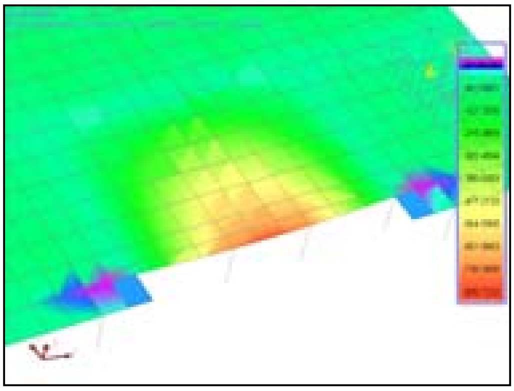


Figure-11: Principal compressive hoop stresses in the concrete floor plate

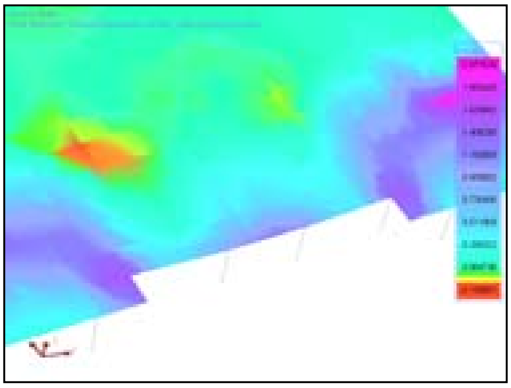


Figure-12: Principal tensile radial stresses in the concrete floor plate

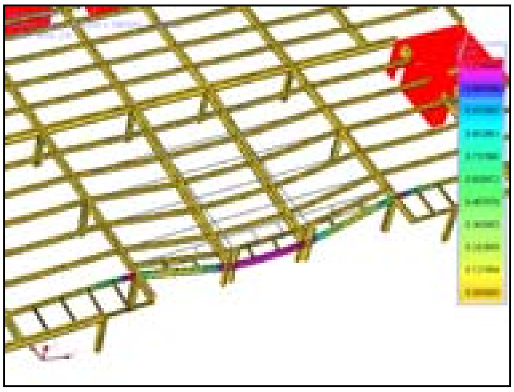


Figure-13: Plastic hinge formation in the main catenary framing element

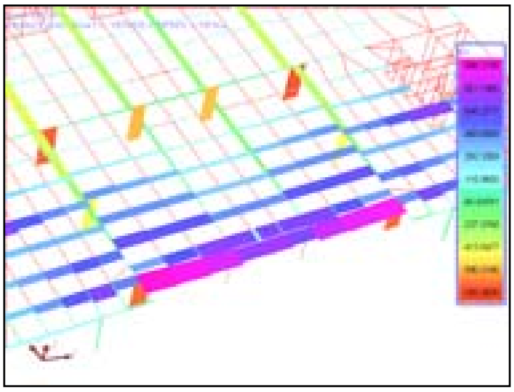


Figure-14: Catenary force diagram

The result of the analysis provides information for design and detailing as follows:

Beams & Girders:

Except in a few locations where beam sizes upgraded to meet the member force demand, majority of the floor framing sections were adequate.

Connection:

The analyses show that the beams functioning as catenary members need to have full capacity connection throughout their entire span. Therefore, connections of perimeter beams and interior girders were required to be upgraded to full plastic capacity. The secondary beam connections remained as simple shear connections, however, their capacities were checked to insure adequate transfer of the catenary axial forces.

Slab:

Compressive strength of concrete slab was adequate for compressive hoop stress demand. The tensile capacity of the slab system was enhanced by upgrading the mesh reinforcement and additional rebars at only critical locations.

Steel member strain:

Member strain was limited to 5%.

Floor Deflection:

Depending upon the location and the column removal scenario, the floor deflection varied from 250mm to 900mm.

The combination of the concrete cores, the floors connected to them and the continuously-designed perimeter frames creates a robust three-dimensional system that allows the structure to redistribute the forces in all directions, preventing a progressive collapse scenario resulting from the loss of columns. Of course, in this extreme event the structure will go into the plastic range with large deformations creating the required membrane and catenary forces to stabilise the system.

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