An Overview of Progressive Collapse in Structural Systems

By

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ABSTRACT

It has become evident recently that abnormal loads need to be considered in the design of structures so that progressive collapse can be prevented. Building collapses such as the Ronan Point, Alfred P. Murrah, and World Trade Center have shown the catastrophic nature of progressive collapse and with an increasing trend towards more terrorist action in the future, it is clear structural design must include progressive collapse mitigation. The most critical abnormal loadings that have potential to cause progressive failure are blast and impact. These loads are impulsive and dynamic in nature with the potential to induce destructive forces, and to further complicate matters is the random nature of occurrence which makes it difficult to predict adequate levels of design. Much research has been conducted over the past several decades, but to this day very little standardized language has been published to help designers create progressive collapse resistant structures. What is known is that robust structures can be built economically by following a general design philosophy of redundancy, ductility, and overall structural integrity. Reinforced concrete structures are especially well suited for resisting progressive collapse by specifying steel reinforcement detailing such as continuous top and bottom reinforcement, close spacing of stirrups, strategic locations of splices, continuous reinforcement through joints, and designing slabs for two-way action. Steel structures have good ductility, but connection detailing is usually the weakest point and requires special design, such as the use of the SidePlate™ connection. Regardless of the type of material used, the design should strive for a uniform, regular layout of the structural system with limited span lengths and close spacing of beams and columns. Perimeter defense systems should be employed as this decreases the threat of an abnormal loading. Since there has been little consideration of extreme loadings, existing structures may be inadequate and require retrofit. Although more difficult, it is possible to achieve improved progressive collapse resistance through the use of externally applied retrofits, such as concrete encasement or the application of composite polymer materials.

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1. INTRODUCTION

Structural engineers are facing new challenges in designing safe structures due to the increase in terrorist actions carried out on landmark buildings which has the potential to cause great destruction, damage, and danger to people. As designers, engineers are tasked with understanding all the possible loads that a building may encounter in its life and ensuring that the structural system will remain standing and ensure the safety of those inside. Abnormal loadings in the past were never considered during design, but an alarming string of events, mostly terrorist, have awakened the need for special considerations for potential targeted buildings. It is virtually impossible to predict what exact extreme load may be induced on a building, therefore when designing for structural integrity the most important consideration is progressive collapse. Progressive collapse results when a localized failure spreads to a larger portion of the structure. Several examples will be given of progressive collapses that occurred in structures due to abnormal loading. Such a failure is catastrophic as collapse occurs in an instance, not allowing time for inhabitants to escape. There are certain details regarding design and retrofit of structures to resist progressive collapse that should be followed, especially for materials such as concrete and steel. This paper will present important design guidelines to follow when designing structures against progressive collapse based on past collapses and recommendations developed by researchers.

1.1 WHAT IS PROGRESSIVE COLLAPSE?

Progressive collapse was never as public topic of an issue until the landmark event of September 11, 2001 World Trade Center attack. The destruction and catastrophic nature of a progressive collapse was viewed by millions that day as two of the most recognizable buildings in the world came crumbling down in a matter of seconds. There have been numerous other examples of structural collapses, but none so visible. Such a failure is a structural engineer's worst nightmare. It is the ultimate goal of the designer to compose a structure that will stand true to the test of time by being able to resist the loads imparted on it and ensure the safety of its usage. Structural engineers are responsible for developing safe designs because any lapse in judgment may result in injury or death. The concept of progressive collapse is starting to be better understood and it is now more essential than ever for engineers to be aware of the abnormal loadings that may initiate a failure, especially since traditional design does not consider these extreme loading scenarios.

Progressive collapse is ultimately the failure of a structure due to so loading condition that gives rise to a more widespread failure of the surrounding members. The following is given by ASCE 7-02 and is the accepted definition of progressive collapse:

> *"The spread of an initial local failure from element to element resulting in the collapse of an entire structure or a disproportionately large part of it"*

Progressive collapse may be likened to the domino effect in a building where the local failure results in a subsequent failure nearby and that failure causes another failure and like a chain of dominoes, once the string is initiated, the collapse continues until its momentum is arrested. Perhaps a more pertinent example is a house of cards. As can be imagined, the removal or failure of one of the load carrying cards will result in the entire house collapsing due to the structure's loss of load transfer across the fallen card. This type of failure is catastrophic due to the speed in which it spreads through the structure in a matter of seconds. It should be noted that progressive collapse does not necessarily involve collapse of the entire structure. In many cases, such as the Ronan Point collapse, a small portion of the overall structure failed. The disproportionality of a progressive collapse refers to the magnitude of the initial failure as compared to the structure's final damage. Most full collapses have occurred during the construction phase due to improper sequencing or accidental overloading of members. This report will focus on progressive collapses that are caused by abnormal loads, such as blast or impact.

1.2 BUILDING COLLAPSES

To better illustrate the importance of considering abnormal loading and progressive collapse, several building failures will be detailed. These case studies reveal the destructive nature of progressive collapse as well as the need to consider abnormal loadings. It is important that engineers learn from past failures in order to prevent them from repeating in the future.

1.2.1 Ronan Point Collapse

The first major progressive collapse to raise concerns about the way buildings are designed was one of the Ronan Point apartment buildings in London. This structure was a 22-story, 210 ft tall building composed entirely of precast concrete elements (Aquino et al). Construction occurred from 1966-1968 and the design was covered by building code developed in 1952. Due to the time period and outdated code a very poor design was developed and shoddy construction prevailed due to the swiftness desired in the construction time. London was in a stage of furious rebuilding due to the destruction from the Second World War. All the structural components were precast concrete elements that were fabricated with slots and could be bolted together. This structural system was known as the Larsen-Neilsen system, composed of entirely factory made precast members and was designed to decrease construction time (Rouse & Delatte). This system was entirely load bearing with each floor supported directly by the walls and columns underneath.

On May 16, 1968, an explosion on the 18th floor caused by a gas leak blew out an exterior wall in the corner of an apartment. The loss of this wall caused floors 19-22 above to collapse down onto floor 17, which could not carry this added weight and also gave way, creating a domino effect all the way down to the ground. Four people were killed and seventeen were injured. This progressive collapse occurred only in one corner of the building, but was severe enough to raise serious questions about the integrity of the building design and construction. The ensuing investigation discovered that the explosion was relatively small, on the order of 10 psi (Aquino et al). Tests on the walls of the buildings uncovered that they could be blown out at minimal pressures of approximately 3 psi.

Figure 1: Ronan Point Failure (Rouse & Delatte)

The Ronan Point collapse was a case in which a very poor structural design with little redundancy for redistribution of forces allowed for a progressive collapse. It turns out that the exterior walls were not even strong enough to resist possible wind loadings in the area, and as a result the building code was re-evaluated and updated. Although the blast loading was not extremely large or terrorist, it created awareness of the potential danger of blast loading and the need for redundancy in building design to guard against progressive collapse. The joint detailing was very poor and deserves special attention. The precast panels had slots so that adjacent pieces could be bolted together and reinforced by dry-packed mortar (Rouse & Delatte). This type of joint is not ductile for there is no consideration for load reversal and continuity across the connection in the case of a bolt failure (See Figure 2). It is also important to note that there was no structural framing system to offer alternative load paths. The result was a building that was not integrated or continuous and became a prime example of the need to consider progressive collapse in structural design.

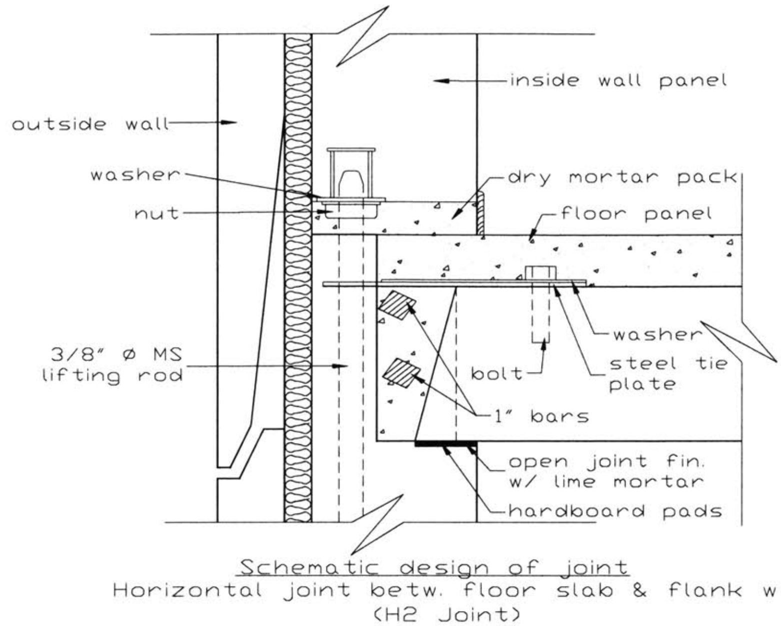


Figure 2: Typical Ronan Point Connection (Rouse & Delatte)

1.2.2 Alfred P. Murrah Federal Building

One of the more well-known collapses occurred in 1995 at the Alfred P. Murrah Federal Building, otherwise known as the "Oklahoma City Bombing". On April 19, 1995 a truck bomb carrying approximately 4,000 lb of fertilizer explosives was parked next to the building and detonated, causing a progressive collapse on the side of the building facing the truck (Longinow & Mniszewski). Not only was there great damage to the building and surroundings, but also 168 people were killed and over 800 injured. Many who died were trapped in the part of the building that collapsed.

Figure 3: Oklahoma City Bombing
(http://techunl.net/murrah/default.htm)

The Murrah building was designed in the early 1970's as a nine-story reinforced concrete US government facility (Aquino et al). It was designed as a simple moment frame structure with columns connected to floors by transverse beams. A relatively short and wide building, the plan dimensions were 220 ft east to west and 100 ft north to south. Of special note is the front atrium of the building where a large transfer girder of 40 ft span allowed for a large column-less entranceway. This girder supported three sets of columns extending through floors three through nine. The truck bomb was parked directly in front of the columns supporting the transfer girder, at a stand off distance of 16 ft. The force from the blast blew out the columns supporting the transfer column, causing the girder and the floors above it to cave in and initiate a progressive collapse. Essentially the entire northern half of the building fell to the ground as a result.

A federal investigation of the collapse determined that the moment frame structural system of the building was not adequate to provide load redistribution resulting from removal of a first floor column (Aquino et al). The building had been designed under the ACI code from 1971, which did not require any consideration for blast, earthquake, or extreme loading conditions. Modern codes require some extreme loading cases to be considered, such as removal of one column for redundancy. The investigation committee determined that had a 1st floor column been removed from the Murrah Building, the loads distributed laterally to the adjacent columns would have exceeded their yield moment and shear. This brings up an important blast resistant design guideline of ductility and redundancy in order to prevent progressive collapse. If this building had been designed under recently developed moment frame detailing, such as those used in seismic regions, the collapsed area would have decreased by approximately 50%. This shows that current design guidelines have improved performance under blast loadings, but still the chance for progressive collapse exists and further action must be taken to better understand the requirements for its prevention. This is especially true for federal government facilities which typically are at a higher risk for blast loading and must be designed with special consideration for progressive collapse.

1.2.3 World Trade Center

On September 11, 2001 two Boeing 767 aircraft were flown into the World Trade Center towers in a terrorist attack. The towers were both 110-story tall steel buildings approximately 1365 ft tall and 208 ft wide. The structural system is described as having a strong steel-framed central core and closely spaced steel tube columns around the exterior perimeter. Spandrel beams between the exterior columns were quite deep, having thickness of 4 ft 4 in (Aquino et al). Floors were supported by open web joists and were framed to provide two-way plate action. Outrigger trusses provided lateral stiffness to the building and helped tie the exterior sides to the interior core. The north tower was attacked first by a plane traveling at approximately 470 mph between floors 94 and 98. The south tower was struck soon after between floors 78 and 84 at an approximate speed of 590 mph.

The damage imparted on both towers at the instant of impact was quite substantial due to the size and speed of the planes which created large impact forces due to the size of the momentum. It has been reported by NIST that airplane debris passed entirely through the building and pieces were found on the other side in streets and building-tops. As a result of the impact, a large portion of the framing system was damaged including the central core. The most damage occurred at the point of impact where exterior columns were

Figure 4: World Trade Center Attack
(http://www.loc.gov/exhibits/911/images/01810r.jpg)

severed by the planes. After impact large fire balls burned unabated that were strengthened by the jet fuel. NIST has reported that fireproofing in these areas was knocked off by the airplane impact and the steel members were softened by the burning fires. The fires coupled with load redistribution in surrounding members ultimately triggered the collapse of the towers due to an overall sagging effect of the floors which caused the exterior columns to buckle (NIST). Both buildings tumbled to the ground in a domino effect in only several seconds.

From a structural standpoint both buildings performed quite well in that they did not collapse for almost an hour after sustaining major damage due the impact of a 767 airplane and the subsequent unchecked fires. The structural system of the towers is described as a highly redundant design which was able to redistribute the extra loads induced on surrounding columns and beams created by the loss of a significant number of columns. Although the buildings fully collapsed in progressive manor, the failure is not a true progressive collapse because a large area in the building failed and collapse did not result directly due to the impact load. It is generally believed that fire caused the collapse and that the buildings resisted the initial localized failures caused by the impact loading. It is important to make note of the qualities of the WTC design that allowed it to perform well under a massive impact load. The towers were designed with close spacing of perimeter columns, deep spandrel beams, hat trusses that stiffened the central core, floors that allowed two-way plate action to better redistribute loads, and was designed for abnormally large wind loads. All of these qualities allowed for a robust, redundant design that provided alternative load paths because the structural system acted compositely with different sections allowing it to compensate for a weakened section. Even though the World Trade Center towers collapsed, their design may be studied as a good system for resisting progressive collapse.

1.3 THE NEED TO DESIGN FOR PROGRESSIVE COLLAPSE

There is a need to design against progressive collapse due to an increasing trend of terrorist action against important facilities. Of particular interest are U.S. embassies which exist all over the world and in countries where anti-American sentiment is high and the threat of attack is very real. It was reported in the 1970's that 5 to 14 attacks per year occurred against U.S. facilities (Gurvin & Remson). This led to federal involvement in the form of an advisory board that developed design recommendations for U.S. embassies against blast events. Among their recommendations was the use of reinforced concrete as the structural material of choice and two-way slabs for additional redundancy. On August 7, 1998 two embassies in Nairobi and Dar es Salaam were bombed on the same day. Both buildings were reinforced concrete that fared quite well and did not progressively collapse. The Nairobi embassy was designed under the recommendations of the federal board and displayed good redundancy and ductility, but considerable damage and injury resulted from non-structural damage inside the building from such items as windows, ceilings, and furniture that were propelled through the air by the blast. These attacks show the effectiveness of simple guidelines which can prove to greatly enhance a structure's performance under blast loading and more importantly prevent progressive collapse.

The examples of the US embassies in Africa show improvement and increasing awareness of progressive collapse, but there still is a need to better understand this mechanism of failure and the potential threats. The Alfred P. Murrah building was a federal building not much older than the African embassies, but it did not perform well in a blast loading. Modern buildings are better designed against progressive collapse due in most part to more widespread seismic code requirements. Seismic design calls for ductile structures which provide an order of resistance to progressive collapse, but it must be clear that it is not sufficient by itself to limit a progressive collapse. It is also important to identify deficient buildings that require retrofit. This in turn needs retrofitting schemes to be developed for built structures, which is not easy due to accessibility to structural members. With terrorism on the rise and no end in sight, it is likely that blast events will continue to be employed on certain structures. Safety and structural integrity must be ensured by endorsing progressive collapse design and continuing research of abnormal loads.

2. DETERMINING POTENTIAL BLAST HAZARDS

Structural engineers have well developed methods for designing structures based on criteria such as loads, usage of the structure, client needs, code guidelines, etc. Most structures are designed for strength based on load combinations that are supposed to simulate the worst case loading scenario during its lifetime. When it comes to progressive collapse considerations, there is much uncertainty in regards to abnormal loads that cause the collapse. As the term suggests, these low-probability loads are abnormal and therefore extremely difficult to predict. Terrorist acts may be predicted to some extent based on the importance of a structure, but the type of threat, size of attack, and time of occurrence are all unknowns. Some blast loadings may be the result of a system malfunction or occupant misuse, such as an internal gas or chemical explosion. There are many different scenarios that may bring about abnormal loading to a structure, so many that it is not possible to predict every single hazard that may affect the integrity of a structure. Hazards are typically classified as pressure loads, impact, or deformation-related (Ellingwood). Loads that may be considered abnormal include explosions, detonations, severe storms, vehicular and aircraft impact, missile impact, and fire. All of these are possible of causing progressive collapse, but this report will focus on those loads that are classified as blast and impact. A blast load will be discussed in more detail in following sections.

2.1 TYPES OF BLAST LOADINGS

Blast loads are described by the release of a large amount of energy in a short period of time that imparts an impulsive dynamic action. This type of load can be very destructive depending on the blast's strength and positioning relative to the structure. The nature of such loads is especially prone to initiating a progressive collapse because the structural system may be loaded in a way that is contrary to the intended functionality of the structure. It is this reason why blast loads will be described in detail in regards to all of the possible abnormal loadings that may be considered in progressive collapse design. The blast loads that will be detailed include explosions, detonations, bombs, as well as structural impact.

Most blast loads impart pressure through a shock wave releasing a great amount of energy in a short period of time. Explosions, detonations, and bombs all fall under this category. Examples of this type include both military and homemade explosive devices, vehicle bombs, suicide bombers, gas explosions, and chemical explosions. A bomb or explosive is classified by its charge weight equivalent to TNT (trinitrotoluene). The number of occurrences of bomb incidents has been steadily increasing in recent years due in a large extent to terrorism. Important buildings such as national embassies are particularly susceptible to attack. According to the FBI, more than 250 bombing incidents against buildings occurred in 1997 (Ellingwood). A recent press release from the State Department identified 650 international terrorist attacks in 2004 (Mohammed). These statistics, along with recent noteworthy attacks such as the Oklahoma City bombing and World Trade Center attack, show that explosive blast loads need to be considered in building design. The difficulty with design is that the actual size of the explosion depends on a number of factors, including the type of bomb and the expertise of the maker. To further complicate design is that information on structural behavior in response to explosions is sensitive material that is mostly gathered by the government and is not widely available (Ellingwood). Even so there is enough information available from research to give engineers an idea of the behavior of blast loads and expected magnitudes for design purposes.

The World Trade Center attack was one of the most destructive and influential terrorist attacks and has brought about an awareness for the need to consider aircraft impact. Impact is not a blast load, but explosions may result such as what happened after the aircraft blew up upon impact with the World Trade Center. This type of loading is very critical because there is a good chance that the combined effects of impact and successive explosion or fire may initiate a progressive collapse. Impact may also be in the form of a vehicular impact with a building, which is a possibility in an urban setting. Such an impact may cause progressive collapse if a first-floor exterior column is hit. This type of loading is best prevented by designing the site with a reinforced perimeter that ensures a maximum keep-out distance for potential hazards.

2.2 BLAST BEHAVIOR & INTERACTION WITH STRUCTURES

It is important to understand the physics of a blast in order to develop practical guidelines in designing a blast resistant structural system. A blast may be any explosion that results in a sudden release of energy. The blast can be thought of as a shock wave that moves away from the source with an intensity and direction dependent on the distance from source, orientation with respect to the structure, and geometry of the obstruction in the path of the wave (Goschy). Blasts create a large amount of energy due to detonation of chemically reactive systems that occupy a very small volume of gas at very high pressure, which creates high temperatures and fuels the expansion of the blast wave. Blasts will be categorized as either internal, external, or explosions in the air. The basic strength and behavior of a blast can be derived from the charge weight and standoff distance. Charge weight refers to the bomb size and is reported in terms of an equivalent amount of TNT, while standoff distance is the distance between the source and the target. A blast shock wave of pressure has been studied in the past mostly for military purposes and the characteristics are well-known at this time. There are many factors that will affect the behavior of the blast wave such as the angle of incidence, surface of contact, venting "reliefs" that exist in the structure, closeness to the structure or obstruction, and duration of the blast. It is very difficult and thus impractical for an engineer to design a structure for specific blast loads, only if it can be determined that certain blast loadings are more probable. Therefore blast resistant design is generally an exercise in designing redundant and robust systems that will act in a ductile manner in a variety of abnormal conditions, which generates from understanding the basic properties of a blast load.

One way to prevent progressive collapse is to ensure that blast loads are not induced on the structure by limiting access to or around a potential target. Although unlikely, it remains possible that an internal blast may occur in a structure either by carefully planned terrorist attack or a gas explosion. Internal blasts may be very damaging because they initiate inside the structure and have access to a larger portion of the structural system. Such a blast may be in the form of a planted bomb, suitcase bomb, vehicle bomb in an underground parking lot, and explosion of flammable dusts or volatile liquids. Ignition of blasts from dusts or liquids is usually accidental. Upon ignition or detonation, there is an abrupt pressure rise from the source outward in all directions whose magnitude depends on the charge weight of the blast and presence of vents. Vents, or pressure reliefs, are openings inside the structure that allow the blast pressure to exit the interior. Typical vents include ventilation systems, windows, doors, thin walls, and many more. Without vents the pressure may be as high as 10-15 psi theoretically, which is approximately 7-10 times atmospheric pressure (Goschy). This would be an absolute maximum case because it is inevitable that vents will exist in the structure, therefore a more realistic peak pressure is approximately 3-5 psi. The peak pressure as a result of a blast is reached very quickly, typically within 20-300 ms (Goschy). Once the peak has been reached the pressure will decline relatively swiftly and a negative pressure region will follow as a result of the shock wave passing though the structure (See Figure 5). This negative pressure creates a suction effect which reverses the direction of loading and is an extremely important consideration for progressive collapse design. Blast loads are impulsive and thus very critical because of their sudden release of energy in a dynamic fashion. When modeling a blast load it is more accurate to use a dynamic impulsive idealization of the pulse wave based on the peak pressure and duration of the pulse. Empirical methods have been developed to calculate internal blast pressures and durations, which will be mentioned briefly later in the report.

The most likely source of a blast load is an external detonation since the perimeter usually can not be totally secured and is also the easiest location for an attack. External blasts include bombs, missiles, vehicle bombs, and even gas explosions that occur outside the building. The behavior of the blast wave is similar to that of an internal blast except that the wave must travel a certain distance before reaching the structure and thus the structure will feel an instantaneous jump in pressure once the wave hits the surface. Another difference is that the area over which the loading will act is dependent on the size of the blast, standoff distance, and duration of the loading. Short duration loadings tend to act on localized areas of the structure and typically have a charge weight of approximately 50-100 kilotonnes of TNT (Goschy). In order for a blast to act on the entire structure a large charge weight is needed as well as a long duration which would be in the range of megatonnes of TNT (Goschy). Before the blast front hits a structure, the blast may be modeled as shock waves of overpressure fronts that decrease in magnitude with distance from the source (See Figure 6). Once the shock wave hits the structure, there is an abrupt jump in positive pressure that declines exponentially to negative pressure and eventually dies out (See Figure 5). The duration of positive pressure depends on the charge weight of the blast and typically ranges from 0.01s to 1s (Goschy). The negative pressure phase may have a longer duration but magnitudes generally remain less than 3 psi (Goschy). It is important to note that the blast wave is reflected when it hits an obstruction which tends to magnify the peak pressure felt by the structure (See Figure 7). Maximum reflection occurs with a normal angle of incidence and decreases as the angle of incidence falls to zero. The reflected pressure can be at least twice the incident shock wave pressure depending on the angle of incidence and the strength of the blast (Longinow & Mniszewski). Already having a large magnitude, reflected waves have the capacity to induce very large impulsive dynamic loads to structures that need to be considered for the prevention of progressive collapse.

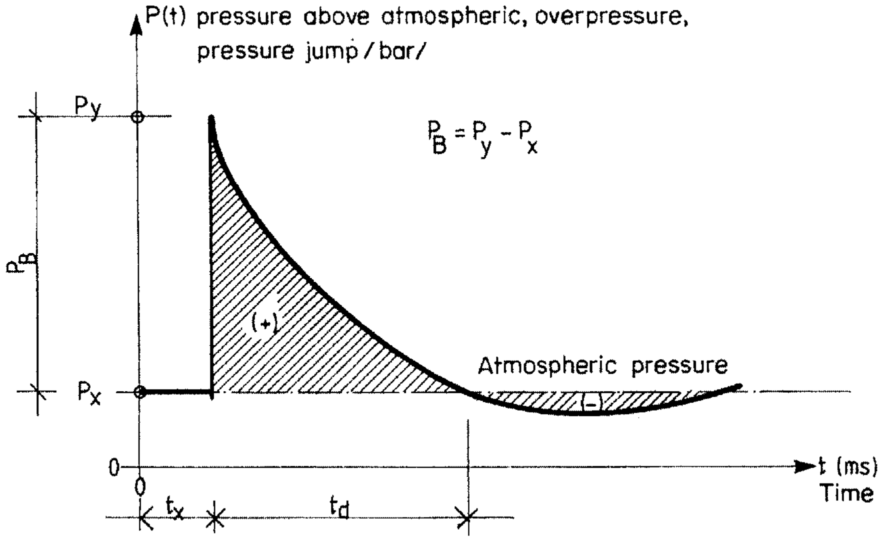


Figure 5: Blast Pressure Wave Relationship (Goschy)

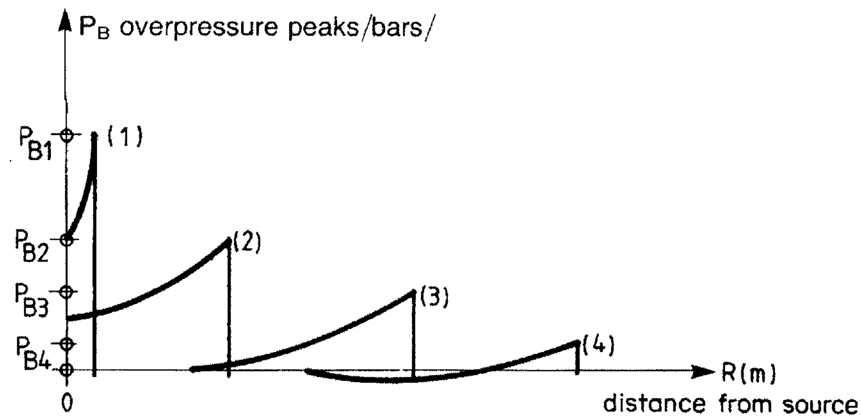


Figure 6: Blast Relationship with Distance (Goschy)

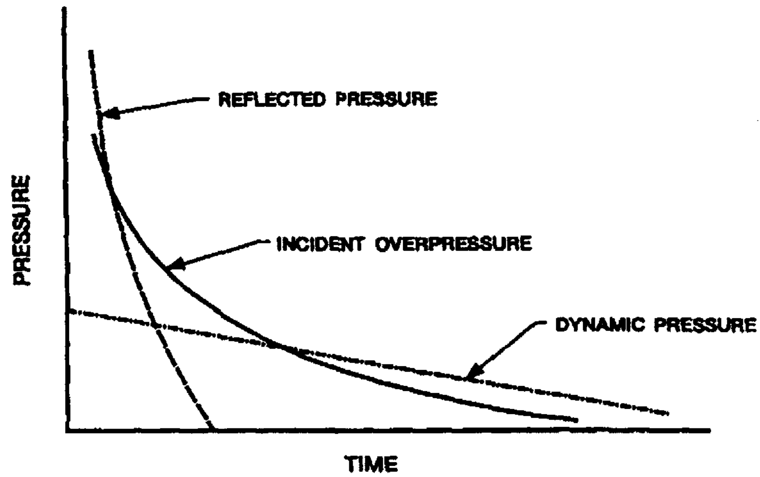


Figure 7: Reflection of Pressure Waves
(Longinow & Mniszewski)

A final consideration for airborne explosions is included for the case of missiles or bombs dropped from the sky and detonated in the air. Such an explosion sends out shock waves in a spherical manner through the air in all directions. The properties of the blast wave are once again similar to that for internal and external blasts, the difference being the nature of the explosion and the route traveled by the wave to the structure. The intensity of the wave is decreasing along the distance from the source to the target and may be mitigated by the atmospheric conditions. The pressure vs. time relationship for airborne explosions is similar to external blasts since the structure will feel an instantaneous jump in pressure once the shock wave hits the surface (See Figure 8). Reflection is once again possible and the pressure induced on the structure will result in a positive and negative phase. This type of blast loading is less likely for conventional structures under terrorist attack but is included here for thoroughness of the understanding of blast behavior.

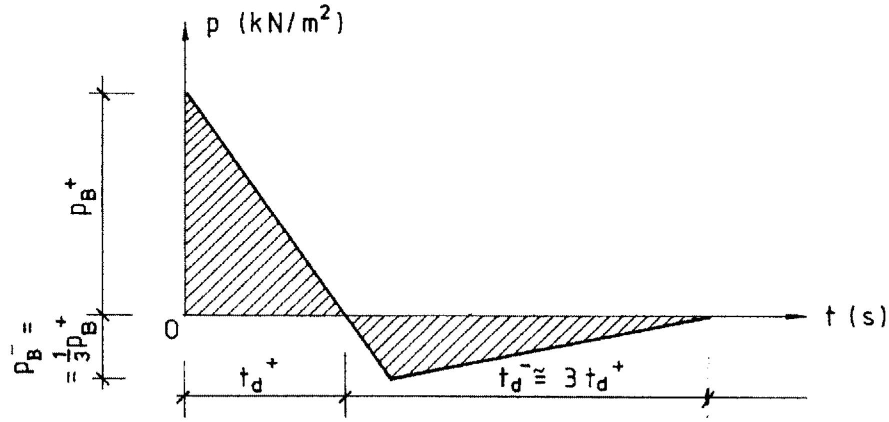


Figure 8: Relationship of Air Blasts (Goschy)

Impact loading on structures needs to be considered because it is a likely scenario for abnormal loading that may cause progressive collapse. It has been shown that vehicular impacts and even aircraft impacts need to be considered, as well as impact from missiles or dropped bombs before they detonate. Instead of a blast wave imparting pressure on a structure, impact deals with a sudden collision that disturbs the energy balance of the system. When a projectile with momentum strikes another mass such as a building, the kinetic energy of the projectile must be dissipated either by deformation of the projectile or absorption by the structure. In most practical cases the kinetic energy will be absorbed by both the projectile and the structure. For progressive collapse purposes it is important to know how much impact the structure can take without yielding or failing. Impact loading may be generalized using Newton's Laws and treating the interaction as two masses on springs (See Figure 9). It is of interest to determine the magnitude of possible impact forces on the structure and the amount of resistance to such loadings the structure may have. Depending on the geometry of the projectile and the location of impact on the structure, the loaded area will absorb the impact by displacing and yielding before failure or a brittle shear punching failure may occur. For progressive collapse prevention it would be beneficial to incorporate detailing to achieve a ductile structure so that impact may be resisted by a structure having a high capacity to absorb energy before failing. It is worth noting that reverse displacement after an impact (rebound effect) may occur due to the storage of energy in the system which can cause severe damage in structural members (Goschy).

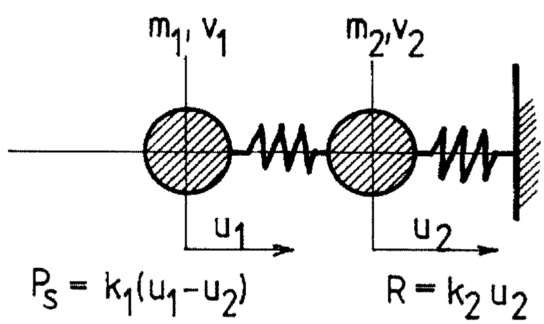


Figure 9: Generalized Spring-Mass Relationship for Impact (Goschy)

Blast behavior for most applicable cases is represented by a sudden release of energy that sends shock waves away from the source and imparts a jump in positive pressure on the structure followed by a negative pressure phase of smaller magnitude. The interaction of this type of loading with a structure is very important because the loads may be amplified due to the angle of incidence upon reflection. It has been reported that this amplification may be twice the original peak pressure of a blast wave. Upon entrance into the structure, the blast wave will decrease in magnitude depending on the presence of vents or reliefs. The negative phase of blast waves creates a suction effect on the structure which reverses the direction of loads and may cause an uplifting effect. This scenario is especially important in floors or roofs that are designed only for gravity loads and may fail and lead to progressive collapse. Thus it is necessary to detail members for multi-directional resistance when blast loadings are a real threat. Impact loading on structures requires that there be a way for kinetic energy introduced in the system to be absorbed without causing failure. Here it is important that the structure be ductile so that it is capable of deforming plastically and absorb a large amount of energy before failing. Brittle behavior may lead to a shear failure where a column could be sheared off, possibly initiating a progressive collapse. The interaction of the structure with an impact loading will be affected by the location and magnitude of the impact and the ability of the structure to resist the momentum in a ductile manor.

2.3 QUANTIFYING A DESIGN BLAST LOAD

Blast-resistant design is not an exact science in structural engineering because of the uncertainty involved in determining the abnormal loads that will act on the structure. As such, very few codes exist that explicitly state what loads the structure must resist for blast-resistance. If such guidelines existed, there is no doubt that they would have to be conservative and would result in extremely expensive and overly fortified structures that no one would want to inhabit. The way to go about quantifying abnormal loads for the design against progressive collapse has been widely debated for many years and there are a few basic methods that are favored by various codes and building agencies. The methods that will be described in this report are the direct, indirect, and alternate load path methods. The direct method is a performance-based design approach in which specific blast loads are computed and applied to the system followed by analysis to determine whether there is adequate redundancy and alternative load paths to resist collapse. The indirect method is a more general approach with the ultimate goal to improve the overall structural integrity of the structural system by performing improvements and detailing that will result in a more robust and redundant system. The direct method is a tedious process involving much iteration and determination of relevant abnormal load threats to the structure, while the indirect method is more widely used since the risk for most structures is low and thus a more general approach without a complicated analysis is in most cases appropriate. Another design method that is endorsed by the U.S. General Services Administration (GSA) is the alternate load path method. This method is more quantitative than the indirect method but does not require as much calculation as the direct method in that it calls for the removal of certain important primary load bearing elements and analysis to determine if the system can redistribute the loads across alternate load paths. Due to the complexity and uncertainty with abnormal loads, there must be a high level of risk involved for a direct approach to be taken. There appears to be a movement towards the alternate load path method because it can be modeled and analyzed by common finite element software without rigorous knowledge or expertise. Also, federal agencies in the U.S. have released new recommendations using this approach since it ensures that a redundant system will be designed.

2.3.1 Direct Method

For high-risk structures that require special design for blast resistance the direct method may be necessary for mitigating progressive collapse. The selection of this method for design must be validated because it requires much effort and advanced numerical and computer modeling. This method may be the only rational choice for retrofitting existing structures because critical members need to be identified since access to structural members is limited (Smilowitz et al). Very few codes have specific guidelines for the direct method because it is very prescriptive and dependent on the conditions unique to the structure. Some European codes included minimum tie forces and abnormal load pressures for load-bearing elements developed after the Ronan Point incident, but such practices have fallen out of favor because they tend to be very conservative and uneconomical. It is better to perform a risk assessment of the structure to determine the possible type of blast loads that may be imparted on the structure, and then proceed to quantify the loads and design the structure. This type of design approach tends to consider one extreme loading case that may result in a few critical members being hardened and detailed to resist failure for that case. As a result, a design is achieved that is contrary to the favored progressive collapse methodology of designing a structure to act robustly with a number of different load paths because the loads will now flow through a few hardened members, decreasing the overall redundancy of the system. Another concern is that the structure is designed for one specific risk, which may lead to unfavorable response to different loading scenarios. For these reasons it can be seen that the direct method is more applicable for cases in which the risk is very well defined or in the retrofit of existing structures where it is not possible to perform general improvements to the structural system.

Being a performance-based design approach, it is necessary to estimate the loads in order to perform an analysis once the risk has been defined. The calculation of blast and impact loads is well known and has been researched for many years. The difficulty exists in estimating the extent of damage to the structure because this requires either empirical research data or very sophisticated numerical simulation (Smilowitz et al). Since test data is not likely to be applicable for each structure, computer models are used in most cases. The impulsive, transient nature of blast loads make them difficult to model in finite element programs and expert knowledge is needed to correctly run the analysis. For an accurate simulation a three-dimensional nonlinear analysis should be run. Failure modes are dependent on the material properties which include geometric nonlinearities, buckling considerations, and linear and nonlinear plastic behavior (Krauthammer & Choi). Very few, if any computer simulations are capable of considering all of these modes of failure, so in most cases analytical methods are used that approximate nonlinear dynamic behavior and are applied to individual parts of the structural system (Smilowitz et al). Such practices are effective for determining the general behavior and response of a blast load on the structure. The approximate dynamic pressure of a blast load can be found by calculating a scaled distance parameter derived from the charge weight and standoff distance of the blast, which is found in the following expression (Carter & Shipe):

$$Z = \frac{R}{\sqrt[3]{W}}$$

Where R is the equivalent distance from the source (ft) of the blast and W is the TNT equivalent charge weight (lb). The pressure is found from charts in the TM5-1300 Army Manual based on the value of Z. There also exist empirical equations that estimate the positive overpressure from external blasts. One commonly used equation for the peak pressure exerted on walls or front faces of buildings is (Goschy):

$$P_B = 6784\frac{W}{R^3} + 93\sqrt{\frac{W}{R^3}}$$

Where R is the standoff distance (m) and W is the TNT equivalent charge weight (tonnes). These approximations can be used to obtain reasonable results about how the structure will respond to the blasts that are determined to be of considerable risk to the structure. Determining impact loads can be approximated with more accuracy because the mechanics involved are well defined and impact loads are usually caused by vehicle or airplane collision. Standard weights and velocities can be obtained and the resulting momentum can be calculated to determine the impulsive force that is exerted on the structure.

2.3.2 Indirect Method

The indirect method is a more generalized approach than the direct method because it does not involve complex numerical calculations and risk assessment to determine the possible loading conditions that might cause progressive collapse. Instead, this approach entails an understanding of the ideals and concepts that improve the resistance of progressive collapse and detailing the structure to improve its overall integrity. This method is more applicable to new design because the designer has free range to detail the system from scratch to create a robust, redundant and ductile structure. The advantage of this approach aside from its ease in design is that it provides a uniformity of compliance based on prescriptive requirements that include minimum joint resistance, continuity, and ties between connected members (Smilowitz et al). Requirements of the indirect method include joints and connections that are able to transfer forces in the case of an extreme loading or member failure. Ways to improve the progressive collapse resistance include the use of internal, peripheral, and column ties, continuous reinforcement for positive and negative flexure, detailing floors to achieve two-way action, as well as joint detailing used to improving ductility for seismic activity. There are many sources available on this method that give recommendations to improving a structure's performance to abnormal loading conditions.

2.3.3 Alternate Load Path Method

This design approach is becoming the method of choice with agencies such as the GSA and Department of Defense endorsing its use after the World Trade Center attacks. Similar to the indirect method, this is a threat independent approach that does not consider specific risks. Its advantage over the indirect method is that it calls for progressive collapse measures to be employed, but also requires the engineer perform analysis on the structure by stipulating various levels of damage by removal of key load-bearing members. This analysis is a way to ensure that alternate load paths exist to redistribute loads around a failed member and encourages redundant and ductile behavior. Another advantage is that the analysis may be carried out using static considerations which significantly reduces the complexity and thus can be performed by simple computer analysis. There are three allowable analysis procedures as specified by the GSA in Unified Facilities Criteria (2005) which are linear static, nonlinear static and nonlinear dynamic. The linear static analysis is the simplest and is based on small displacements assuming linear elastic material. A full static load is applied to the structure upon removal of a member and then analyzed. The nonlinear static analysis assumes nonlinear material behavior including large deformations upon the application of a full static load. The nonlinear dynamic analysis assumes nonlinear material behavior and performs a dynamic analysis for the applied load upon removal of the element. The UFC standard recommends a three-dimensional model because it yields more accurate results, but two-dimensional models are allowed because they are easier to run and tend to be on the conservative side. It should be noted that the UFC guidelines were developed for minimum requirements for progressive collapse and are most applicable to buildings of 10-stories or less (Smilowitz et al).

The guidelines have provisions about which members to remove during the analysis including exterior and interior columns and load-bearing walls. The general rule for columns is for the removal of corner and middle exterior and interior columns on each floor or at any locations where there is a change in geometry or loading condition. See the Figures 10 and 11 for a typical building frame with locations of column removal. Analysis should check to ensure that continuity is retained across horizontal members under which columns were removed. Similar provisions exist for the removal of load-bearing walls. For all analyses, the following load combination is applied to the structural system (UFC 3-2.4.1):

$$(0.9 or 1.2)D + (0.5L) or (0.2S) + 0.2W$$

D stands for dead load, L stands for live load, S stands for snow load and W stands for wind load. A description of these loads can be found in ASCE 7-02. In the case of a nonlinear dynamic analysis, this load combination is applied to every member in the system and upon removal the load of the failed member is doubled and applied to the surrounding members that are affected by the removal to account for impact. The load from 3-2.4.1 is also applied to each member in a static analysis, except that the following load combination is applied to all elements adjacent to and above the removed member (UFC 3-2.4.2):

$$2.0[(0.9 or 1.2)D + (0.5L) or (0.2S)] + 0.2W$$

This load combination is essentially double the previous in order to simulate impact and redistribution for a static condition. Once the analysis is run, the UFC states that the damage limit for an external column removal is that the collapsed area be less than 15% of the floor area above the column, while the collapsed area is allowed to be 30% for an interior column removal. Additionally, the Demand Capacity Ratio (DCR) of each primary and secondary member is calculated from the following equation (Bilow & Kamara):

$$DCR = \frac{Q_{UD}}{Q_{CE}}$$

Where Q_{UD} is the acting force in the structural element and Q_{CE} is the ultimate un-factored capacity of the element. Progressive collapse is said to be prevented when DCR is less than 2.0 for typical structures and less than 1.5 for atypical structures.

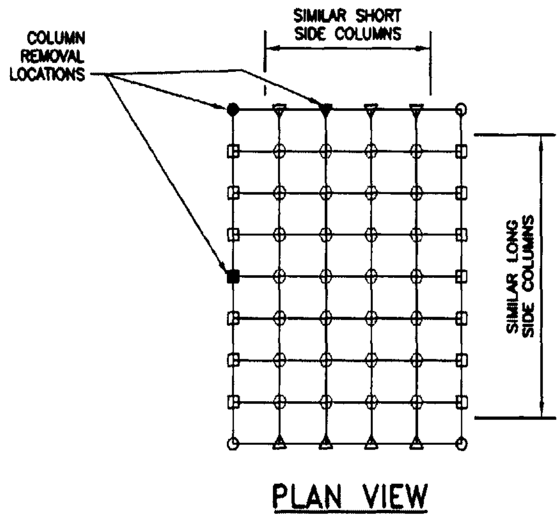


Figure 11: Exterior Column Removal Locations (UFC 2005)

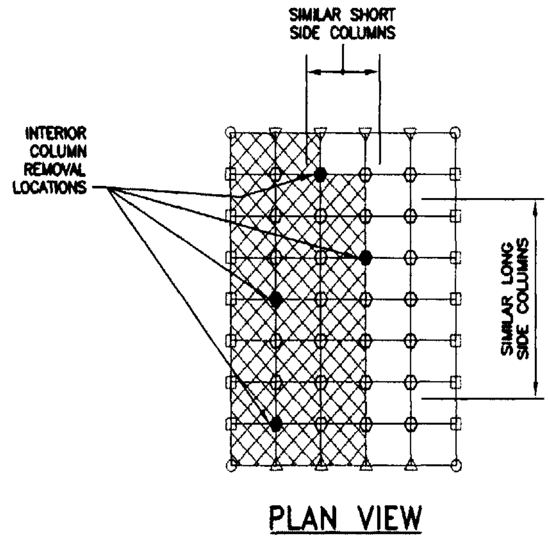


Figure 10: Interior Column Removal Locations (UFC 2005)

It was previously stated that a three-dimensional nonlinear dynamic analysis will yield the most accurate results for an alternate load path analysis. As such, this approach is based on static considerations and may not be adequate for simulating progressive collapse (Krauthammer & Choi). Very few numerical simulations of progressive collapse to this extent have been developed, but a new algorithm attempting to simulate discontinuous problems was carried out recently (Krauthammer & Choi). This method developed a computer program that considered material nonlinearity, geometric nonlinearity, and buckling failure criteria. The alternate load path method was used to model the behavior of the removal of two columns from a typical steel frame multi-story building. The analysis revealed that linear and nonlinear models behaved differently and gave inconsistent results. Also important is that when local buckling was considered, the collapse failure began much earlier than when strain criterion was used (Krauthammer & Choi). These results indicate that buckling and nonlinear behavior should be considered and researched more extensively because they might control when a progressive collapse initiates (See Figure 12). It also shows that the alternate load path method can be quite accurate for simulating a blast loading if a complex model is used, but for most low-threat scenarios, static analyses have been appropriate.

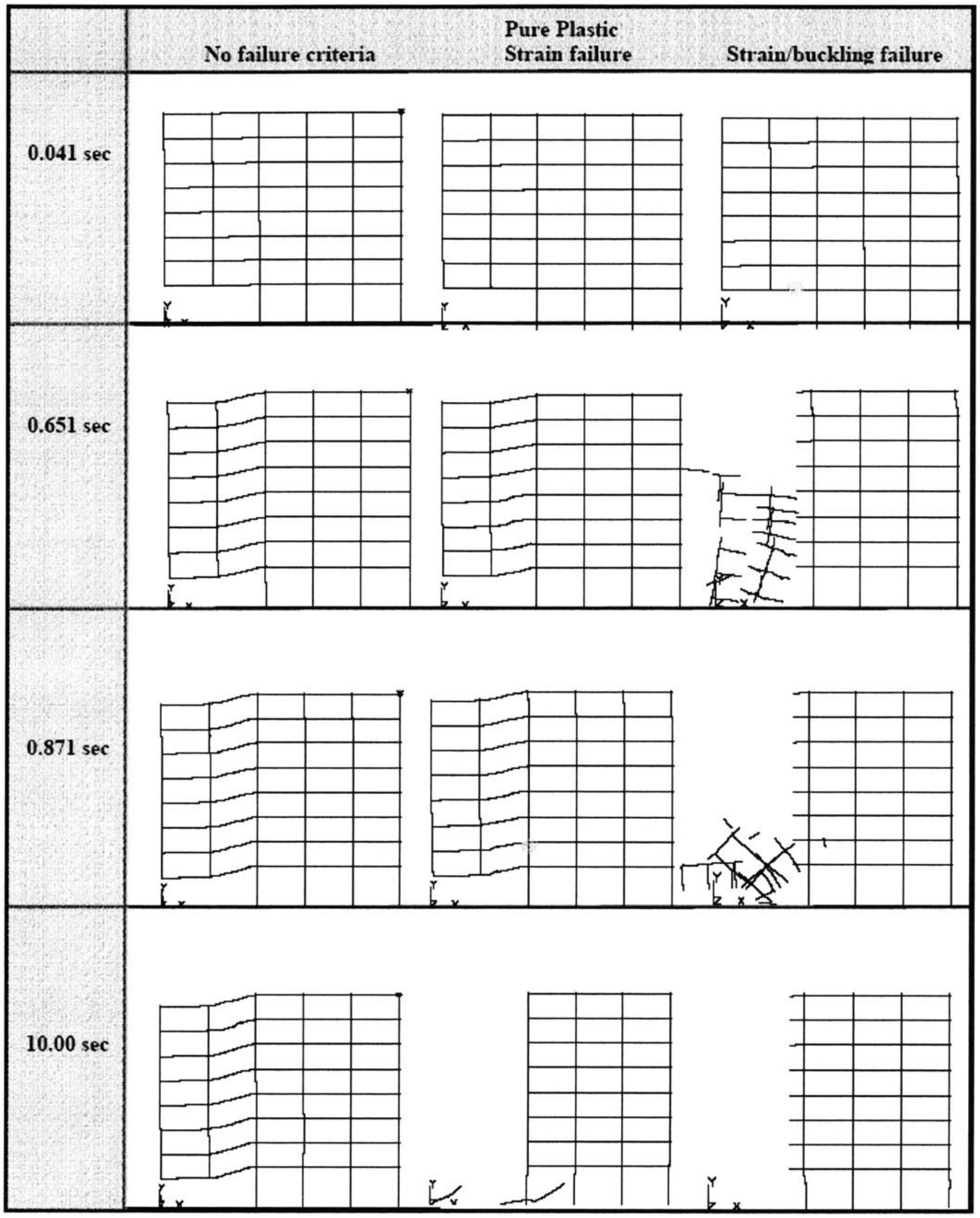


Figure 12: Comparison of Column Removal Failures (Krauthammer & Choi)

2.4 ASSESSING THE RISK

The evaluation of the risk involved for progressive collapse in a structure is dependent on many factors. A good starting point is looking at the definition of risk and identifying the three components that make up risk: hazard, consequence, and context (Ellingwood). Hazard is the harmful potential of the risk and is quantified by estimating the probability of its occurrence. Consequences result from the occurrence of the hazard and are measured by some unit. Finally, context provides a basis for the decision making involved from the risk and is dependent on the individuals or agencies that are affected by the risk. Context is generally different for each person or group because of the way the risk may affect them and what they stand to lose or gain. There are ways to quantify the risk by computing the probability of occurrence based on many constituents that quantify hazard, consequence, and context. Building codes are made to guide designers to construct safe structures that the public can feel safe with. In terms of risk, codes establish a level of risk that is deemed acceptable by the public. Until recently, it was safe to say that the public was not very aware of progressive collapse, but with the recent failures that occurred, there is a much greater demand for buildings that are safe against terrorist attacks and abnormal loadings. Ultimately, the clients will decide what level of risk to design against progressive collapse because of considerations of cost, safety, threat of abnormal loading, design life of the structure, and the functionality of the structure. Obviously the more important a structure is, such as a federal facility or embassy, the more likely it is to be the target of a terrorist ploy. Government facilities tend to have stringent regulations that guide their design, especially for blast resistance. Ever since the early 1980's the U.S. government has determined appropriate risk levels for progressive collapse and developed regulations for design. General building codes do not contain specific regulations such as embassy buildings, but there are generalized statements that pertain to ensuring building integrity. Thus it is up to the client and engineer to develop a level of risk that should be designed for. Standardized structural guidelines pertaining to progressive collapse have yet to be fully integrated throughout structural engineering, but in the past few years there has been a movement to develop these standards. It will take time for these standards to become requirements far-reaching, and thus the assessment of the risk is open to interpretation and should be carefully considered in order to determine the level of resistance needed for progressive collapse mitigation.

3. DESIGN FOR PREVENTION AGAINST PROGRESSIVE COLLAPSE

The design for mitigation of progressive collapse has been a hot topic in structural engineering due to a heightened awareness of blast and terrorist hazards. Many alternatives and suggestions have been proposed by numerous structural engineers and blast experts, and with continued research more alternatives are to be expected in the near future. The challenge exists in making decisions about the best solutions because of the inherent uniqueness that are to be encountered for each project. Also, there is little to no official design standards or guidelines available for engineers to follow to aide their decisions. Instead, the engineer must be well-versed in blast resistance and progressive collapse research in order to have a good understanding of what it takes to build or retrofit a robust structure. There is a push from blast specialists to make this information more public as well as to develop more refined guidelines. Recent efforts are showing progress, but it will take time and greater efforts by federal agencies and academia to urge the need for progressive collapse measures in structural design. This section will summarize a wide collection of guidelines and suggestions in regards to progressive collapse mitigation that have been developed by experts in the field of blast resistant design.

3.1 GENERAL DESIGN GUIDELINES

It is the philosophy of progressive collapse design to pursue the traits of ductility, redundancy, and structural integrity. A ductile system is one that can experience large deformations before ultimate failure, thereby absorbing a lot of energy through inelastic and plastic strain. Such behavior is preferred in structural systems, especially ones induced to abnormal loadings, because generally full flexural capacity will be reached before shear and thus brittle, explosive collapse will be avoided. Redundancy refers to a structure's ability to sustain a local failure without global collapse through alternate load paths, which is critical for progressive collapse mitigation. Also important is achieving a high level of overall structural integrity by designing strong columns, confinement, two-way action in floors, and providing continuity detailing through connections so that the structural elements are essentially tied together and will act compositely to resist and redistribute loadings. There are ways to provide these measures in both new designs and retrofitting schemes. Since steel and concrete are the most common structural materials, separate detailed sections will discuss specific design issues and provisions that are applicable for these two systems. Progressive collapse prevention can also be achieved by a thorough site investigation in order to limit standoff distances for attacks and minimize access to the structure. Such measures should always be considered because they are generally very cost efficient and mitigate the threat of abnormal loading from ever occurring. Other design considerations include façade and cladding selection so as to protect the structural members and to resist fracture so that the chance of flying debris is limited, choosing laminated glass that will better withstand blast shock waves, and limiting the use of transfer girders.

Many researchers have suggested maximizing the keep-out and standoff distances so that threats may be prevented and effect of loads may be minimized, partly because design of the structural members is difficult due to the uncertainty of the loads. The extent of control of the site layout is dependent on the setting of the structure, for instance standoff distances will be more difficult to control in an urban setting where the surrounding areas may already be built up. All potential access areas for abnormal loading must be considered, which include the vicinity of streets, parking lots, loading bays and pickup points, trash containers, and unobstructed spaces where explosive devices may be concealed. The Department of Defense (DoD) released minimum standards for anti-terrorism for buildings which give recommendations for site layout and minimum standoff distances and site protection measures. It recommends that the outside perimeter of the site be controlled by some means, but is not required since this is not possible for all sites. Minimum standoff distances are given based on the level of threat, vicinity to parking or streets, and whether the perimeter is secured. These values range from 33 ft to

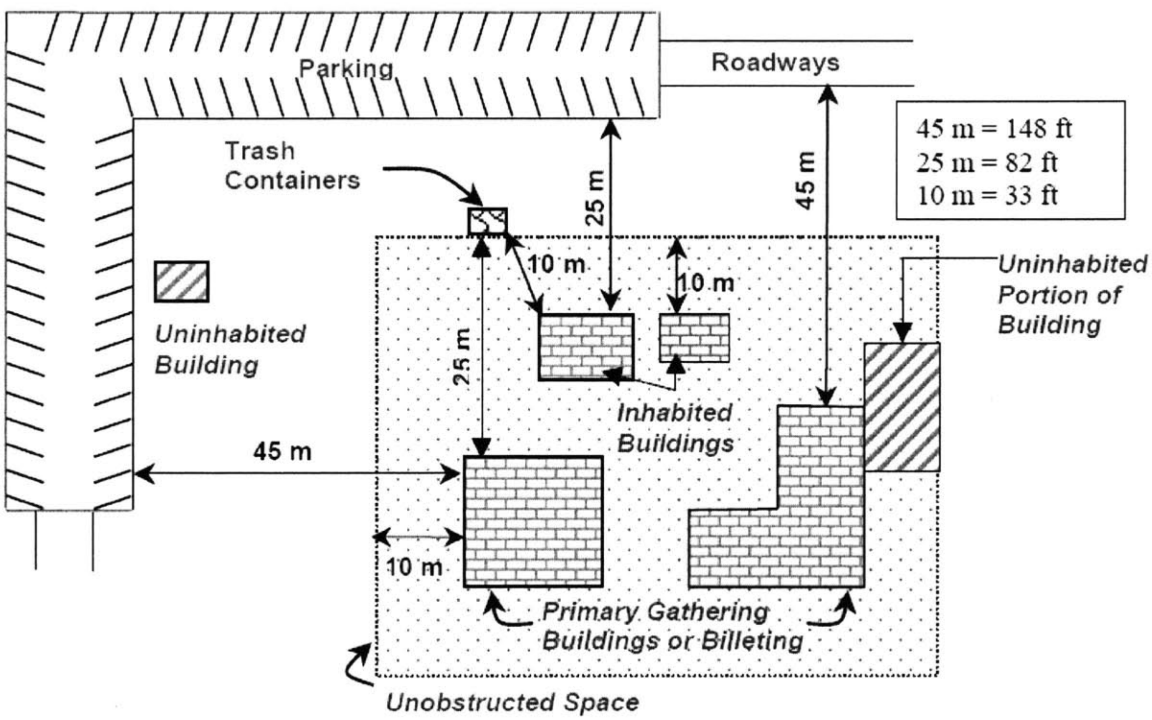


Figure 13: Recommended Perimeter Layout (UFC 2005)

148 ft and a suggested layout is given in Figure 13 (DoD 2001). To mitigate the threat from vehicular impact, the use of anti-ram bollards or large planters around the perimeter that are designed based on the design speeds of the surrounding streets may be specified (Ettouney et al). It is important to limit the vicinity of parking lots and especially underground parking lots because of the threat of car bombs, an example being the first World Trade Center attack in 1993. Depending on the importance of the structure, it may be necessary to provide a certain level of security and control of access to and around the structure in order to prevent close-in attack. These simple and intuitive site considerations can reduce the threat to a structure by making them harder to attack, thereby limiting the chance of abnormal loading and also progressive collapse. The DoD

document provides some standard guidelines, but ultimately it is up to the designer and client based on the site to determine the level of site design.

3.1.1 Ductility

Ductility is a structural response feature that is advantageous in most structural engineering applications because the failure mode shows large deformations before ultimate collapse. When considering abnormal loadings such as blast and impact, strength is not the appropriate design objective because this would result in large, costly, and inefficient systems that still might be susceptible to progressive collapse. Ductility should be the key feature of an efficient blast resistant design that will perform well and remain cost efficient. Obtaining a ductile design depends on the structural system and

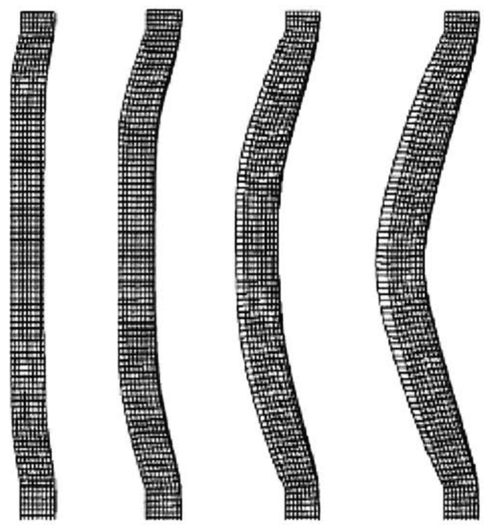


Figure 14: Column Shear Response to Blast Load (Crawford et al)

materials, but there exist well-known guidelines that the designer should follow. Important concepts to keep in mind are the toughness of the material so that appropriate levels of reinforcement and detailing can be implemented to improve ductility, determining the shear and flexural capacities of members and making sure that shear is never the controlling failure mode, and paying special attention to behavior at connections and joints so that full material strength may be achieved before failure. For abnormal loading conditions certain elements may be at risk for load reversals and thus floor slabs, beams, and columns need to be designed for uplift considerations. The extent of these reversed loadings is generally unknown and subject to much uncertainty depending on the risk, but buildings designed for progressive collapse mitigation must look at these possibilities. It is suggested that floor slabs be designed for two-way action so that in the case of a beam or column loss, the slab can span a different direction. A factor of safety should be applied to the beam reinforcing details to sustain such a condition, and in most cases it has been determined that the inclusion of shrinkage and temperature reinforcement in concrete slabs is sufficient (Smilowitz et al). Ductile slabs are obtained by providing both top and bottom reinforcement, and even though it is not required, performance enhancement will be achieved if the reinforcement is continuous throughout the span of the slab. In the case of a column removal, catenary action should occur in the slab in order for loads to bridge the damaged area. Catenary action (Figure 15) is achieved by providing continuous bottom reinforcement completely through column joints (Ettouney et al). Columns should be designed with adequate confinement so that ductile behavior occurs such that flexural capacity is reached before shear, especially exterior columns located near the ground floor. The shear failure mode shown in Figure 14 should be avoided. In particular, direct shear failure is highly unfavorable due to its sudden brittle failure at small displacements. In order to prevent such a failure mode, columns should have good confinement through the use of steel jackets, fiber reinforced polymer (FRP) wraps, spiral reinforcement, and steel columns encased in concrete among others. As long as shear capacity is greater than flexural capacity, ductile behavior will be ensured (Smilowitz et al).

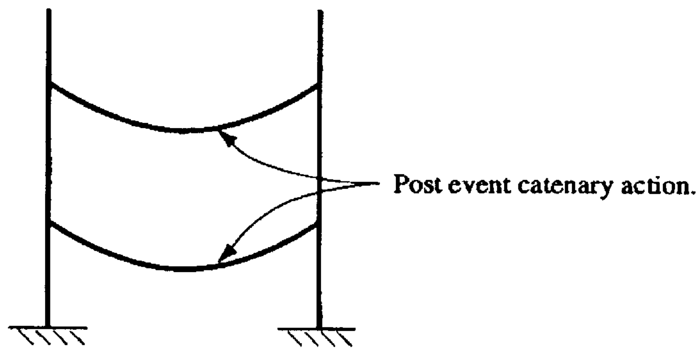


Figure 15: Visualization of Catenary Action (Ettouney et al)

3.1.2 Redundancy

The most common approach to progressive collapse is the alternate load path method in which a column is removed from the system and analyzed to ensure the capability to redistribute loads without collapsing. Such an analysis is meant to create redundant systems that are built with a factor of safety against collapse in the case of an abnormal loading that damages or removes certain load-carrying elements. There should be some level of redundancy for any structural system because of the potential for collapse upon some extreme loading scenario. In progressive collapse, a higher factor of safety is needed than in routine design. It is vital for a structure to withstand an attack by acting in a ductile and robust manner, thereby bridging damaged areas through alternate load paths. Enhancing redundancy in existing structures may not be easy since it is generally achieved in the overall layout and detailing of the framing of the structural system. A multi-hazard approach is recommended that combines the indirect and direct design methods for enhancing a structure's redundancy (Smilowitz & Tennant). Simple hardening of critical members such as lower level columns may not be adequate for progressive collapse. For instance, a large blast may be withstood by a hardened column, but if the surrounding beams and slabs fail, a progressive collapse will occur regardless of the column's strength. Simply strengthening one element does not add redundancy to a system; it is the introduction of load paths that improves the resiliency of a system. This is a fundamental flaw that many designers fail to understand in design and may lead to structures that are supposedly designed for redundancy, but are essentially non-redundant systems with a few hardened members.

There have been many reported suggestions in order to develop redundant structures that inherently perform better against progressive collapse. Before members are sized and detailed, one measure that improves redundancy is designing a uniform structural support system. A uniform layout results in a regular arrangement of elements and joints, allowing the designer to create a greater overall continuity of the individual members to act compositely in resisting a local failure. It is less than ideal to include overhangs, re-entrant corners, and other discontinuities because loads tend to be trapped and concentrated in extreme events (Smilowitz et al). Another general guideline is to limit the presence of long spans and unsupported lengths by the use of more beams and columns. The more elements there are in the support system, the better the system will be in redistributing loads because of the presence of alternate load paths. In the case of bearing walls, intermediate interior walls should be spaced so as to provide better stability if local failure occurs in sections of the bearing wall. Spandrel beams are highly

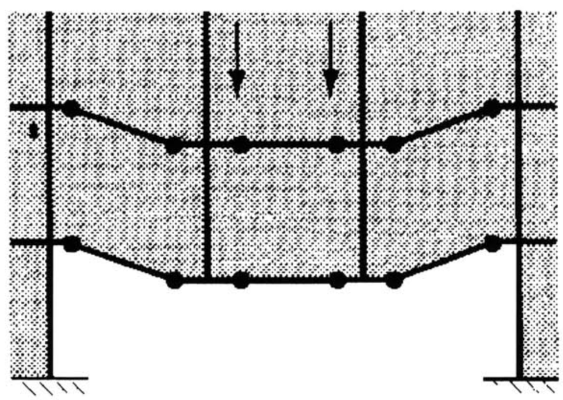


Figure 16: Progressive Failure of Transfer Girder (Ettouney et al)

recommended around the perimeter framing to help redistribute loads around columns (Smilowitz et al). The use of continuous beams and girders is preferred because of reduced deflections and a greater ability to withstand a column removal. Indeterminacy, although not a direct method of increasing redundancy, does provide more robust resistance to extreme loads. This can be obtained by specifying moment resisting connections when simple connections may only be necessary, which will improve the overall continuity and composite action of the structure as well as improve the resistance of the connections to abnormal loading conditions by allowing the formation of plastic hinges. Transfer girders should not be used when there is a chance of progressive collapse since they lack redundancy (See Figure 16). If they must be used, a very detailed analysis of the connections at each end of the transfer girder must be conducted to properly design the strength needed to prevent progressive collapse (Ettouney et al).

3.1.3 Structural Integrity

The previous two sections of design guidelines have stressed the importance of good overall structural integrity in order to resist progressive collapse. There are many modifications that can made to a structure to improve its structural integrity against progressive collapse, which can be summarized by special detailing at connections,

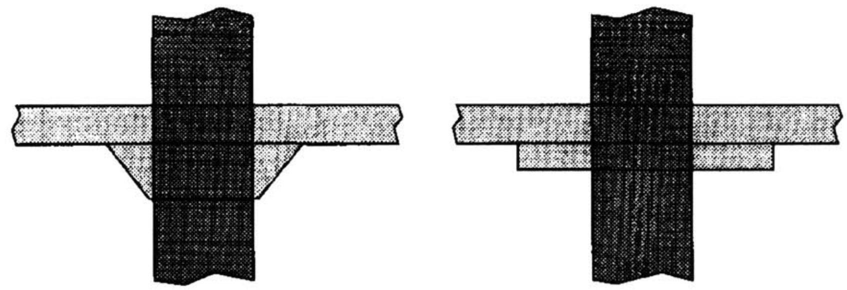


Figure 17: Punching-Shear Prevention Details Near Columns (Ettouney et al)

designing for extra shear capacity, designing columns for good confinement, limiting span length of beams and slabs, considering two-way action and load reversals so that there are alternate load paths and a high level of redundancy exists. Connections should be designed to be stronger than their surrounding elements in order for full flexural capacity to be reached and also so that in the event of a blast load, elements can resist rip-out forces and bridge damaged columns. In order to obtain such behavior, it is recommended to provide continuous reinforcement through columns and include horizontal and vertical ties that are integrated to work together and hold primary elements of the structure together (Dusenberry & Jeneja). As was stated earlier, shear failure modes must be avoided and measures must be implemented. Detailing is very important, especially for preferred system response. Slab-column connections should include drop panels or column heads in order to resist punching-type failure around joints as shown in Figure 17 (Ettouney et al). The designer should recognize that the most likely location of abnormal loading is usually the lower levels on the exterior and design those members accordingly so that progressive collapse may be mitigated. Buckling and stability of columns needs to be addressed if beams or slabs are removed which increase a column's unsupported length (See Figure 18). It has been recommended that columns be designed to withstand buckling for two or three stories of lost lateral support (Smilowitz et al). Other considerations that can improve the structural integrity include the use of lightweight cladding so that the overall weight of the structure is reduced, do not allow exposed columns, provide some type of covering around critical columns to insulate the impact of the loading, and incorporate the lateral load resisting system with the progressive collapse resistance. This can be done by aligning shear walls to produce rigid diaphragm action of the structure, or employing a well-distributed lateral load resisting mechanism throughout the structure that will improve the overall structural integrity in response to a blast load (Ettouney et al).

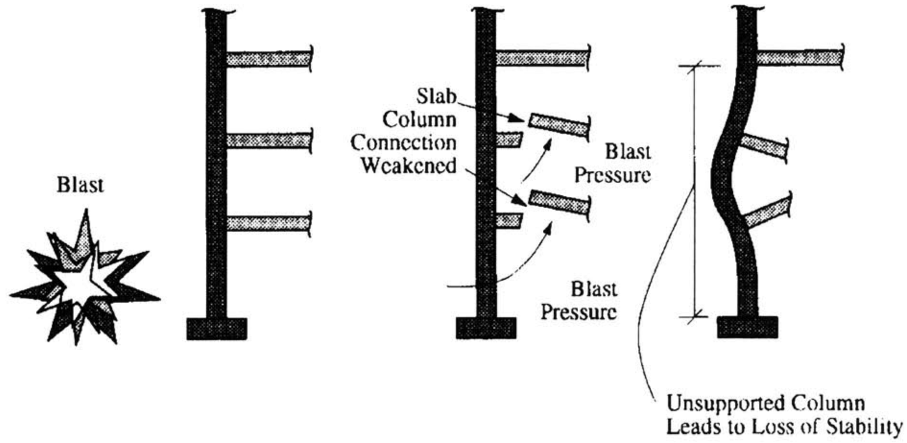


Figure 18: Column Stability Loss from Blast (Ettouney et al)

3.2 DESIGN ISSUES IN CONCRETE STRUCTURES

Reinforced concrete is one of the most widely used structural materials in the world and has been used effectively in blast resistant design. Making concrete resistant against progressive collapse is possible by a variety of measures, the most fundamental being the detailing of steel reinforcement. Un-reinforced concrete is not a good material because of its brittle behavior and low ductility, but the presence of steel reinforcement is capable of producing very good strength and ductility. As with any material, there are advantages and disadvantages for its use. A lot of research has been done in recent years developing retrofitting schemes for progressive collapse mitigation with reinforced concrete structures. A review of some retrofitting schemes as well as guidelines for new construction will be given in the next few sections.

3.2.1 Strengths & Weaknesses

It has been considered for a long time that reinforced concrete is the best material for structures that are expected to withstand blast loads. This stemmed from the military's use of concrete for explosion resistant structures such as military bunkers. Much research was conducted years ago by the military to study the physics of blast and determine concrete's response. They determined that an adequate level of ductility could be achieved in reinforced concrete while also providing better resistance to the thermal effects of explosions than metals. Another advantage of reinforced concrete structures is its greater mass over steel which is beneficial at resisting the dynamic forces induced from a blast load. Since these forces are impulsive in nature, they are characterized as having a very high frequency. A massive structure will have a low natural frequency due to its large mass; therefore a high frequency blast load will most likely not be able to excite detrimental modes that could cause damaging dynamic response. Another advantage of reinforced concrete is that members generally are not slender and less susceptible to buckling. This is beneficial for columns in the event of a floor loss where the lateral bracing is lost and the column's unsupported length increases. Such a scenario is ripe for progressive collapse and it is necessary for columns to be designed with consideration of stability.

How well a reinforced concrete structure reacts to an abnormal load condition depends greatly on the designer's precision with detailing the reinforcement and the quality of the construction. Attention to detail must be taken in order to ensure that shear capacity is adequate, flexural behavior occurs that allows formation of plastic hinges at moment connections, and stirrups and ties are laid out to increase confinement and tie together compression and tensile reinforcement. If the spacing is not correct or reinforcement is not provided top and bottom, an abnormal loading may induce a brittle failure which could initiate a progressive collapse. It is imperative for quality control measures so that the construction of the concrete is to the level expected by the designer. Cast-in-place concrete is preferred because there are more options available in regards to reinforcement layout, but it is also more susceptible to irregularities since the quality is dependent on site conditions. There are other alternatives such as precast concrete sections and prestressed concrete. Precast concrete is used because it results in less construction time, but is not preferred for progressive collapse prevention because it results in a structural system that is very susceptible to collapse. The Ronan Point collapse was a perfect example of the weaknesses of precast concrete. It is possible to provide collapse resistance with precast systems if the members and connections are designed for blast behavior. Prestressed concrete is also susceptible for progressive collapse because of the brittle behavior displayed when stresses exceed the elastic limit. Unless modifications are made, prestressed concrete systems should be avoided.

3.2.2 Suggestions for New Design

The most effective structural systems resistant to progressive collapse are possible when design measures are involved from the beginning. It is much more difficult to upgrade existing structures because there are many more constraints and issues that inhibit the design alternatives available to the engineer. Also, a more resistant structure will be attained if blast-resistant measures are applied throughout the structure since progressive collapse is a phenomenon that is best restrained by overall structural integrity that works together to bridge over damaged areas and redistribute loads. This is can be achieved efficiently in a new design when the general design guidelines described earlier are applied to the structure. The first step in developing a resistant progressive collapse structure is to select a uniform, regular layout of the structural system such that continuity can be achieved. Next the main supporting system should be selected that displays redundancy and ductility, such as a moment frame system. Once these decisions have been made attention must be placed on detailing the columns, beams, and slabs for ductility, redundancy, and overall structural integrity. For concrete structures, connections and splices are very important in response to an abnormal loading and there exist guidelines in the ACI 318-02 that may be followed. Suggestions for the use of precast or prestressed concrete systems will be given as well.

The layout of a structure is important because it will determine critical areas and paths that may be vulnerable to collapse as well as identifying what detailing options are available. As was mentioned earlier in the report, uniform arrangement of structural elements is recommended because irregularities tend to trap loads which lead to stress concentrations, also preventing certain areas from distributing loads around damaged areas. Irregularities usually exist due to architectural features such as special floors and entrances. These sections are especially critical for progressive collapse since there is a general lack of redundancy where large open spaces or high ceilings exist. Although the most elegant design may not be possible, progressive collapse measures can be implemented while retaining a normal structure with a certain level of aesthetics. Typical reinforced concrete building systems include moment frame, shear wall, and bearing wall structural support systems. In general, moment frame systems provide the best collapse resistance because they can provide good levels of redundancy and ductility. Moment frames are at risk for failure in columns due to insufficient shear capacity and concrete crushing at mid-span. Bearing wall concrete structures lack ductility, but has been shown to perform well in blast loads due to their large strength and ability to span fairly large zones of failure (Crawford). This type of structural system perform well if critical areas are hardened or reinforced with an additional back-up wall or small spacing between intermediate walls. To enhance the overall structural integrity columns and beam spacing should be kept to a minimum and long spans should be avoided. Floor slabs should be supported on all sides by beams for added redundancy. In addition to the general building layout, the lateral support system should be integrated to add to the progressive collapse mitigation. This can be achieved by providing monolithically cast spandrel beams around the building perimeter.

Design in progressive collapse has placed emphasis on ensuring column integrity, while it should be noted that this is not the only consideration for design, columns do need special attention as they are especially prone to attack from explosions or impact loads. For reinforced concrete columns, strength and ductility are key because it is desirable for the columns to resist the loads and remain standing, but in the case of a column removal, adjacent columns must have adequate capacity to carry the redistributed loads. This can be checked by performing an analysis of the columns after implementing an abnormal loading to the system using either the alternate load path or direct methods. The column should have sufficient flexural and shear capacity to resist the loads within a certain range of displacement and rotation criteria. The GSA Unified Facilities Criteria has minimum displacement values for various reinforced concrete members expressed as member rotation or ductility. These design values are very general and should not be taken as universal criteria, the designer should use discretion based on the uniqueness of the structure and the level of risk associated. The ACI 318-02 has guidelines for column reinforcement detailing that should be followed for general structural design. Seismic detailing should be included for columns because it increases ductility. Chapter 21 of the ACI 318-02 is a good reference to follow for seismic upgrade. More specifically, columns should be designed to have greater flexural capacity than adjacent beams so that plastic hinging occurs at joints (Smilowitz et al). Flexural capacity is developed through the selection of longitudinal steel reinforcement placed along the outside edges so that the column can resist loads from all sides. It should be noted that steel grades larger than 60 should not be used because ductility may not be ensured (PCI). Once flexural capacity is adequate, column shear must be checked to ensure that it exceeds flexure. Shear resistance is obtained by transverse stirrups, or ties, which may either be hoop bars or spiral reinforcement. Ties enhance shear capacity as well as column ductility by confining the longitudinal reinforcement and allowing larger displacements. Columns should be designed to have high levels of confinement for progressive collapse mitigation by placing a large amount of transverse reinforcement along the length of the column as well as through joint regions. Spiral reinforcement is preferred since it provides better confinement than regular transverse reinforcement (See Figure 19). Checks should be made for diagonal and direct shear capacity so that they exceed the flexural capacity. See Figure 20 for an idealized flexural response and related levels of shear. Low-level and exterior columns may be required to have larger cross-sections so that they can carry larger loads and moments, as well as resist abnormal loadings. Since a large amount of reinforcement is necessary, spacing should be checked to ensure that the steel is not too dense and concrete can achieve adequate bond. Also important is the splice location which should not be placed at mid-span or ends. If the column resistance can not be achieved within a certain limit on cross-sectional size, steel jackets or composite wraps may be used to enhance column confinement and strength.

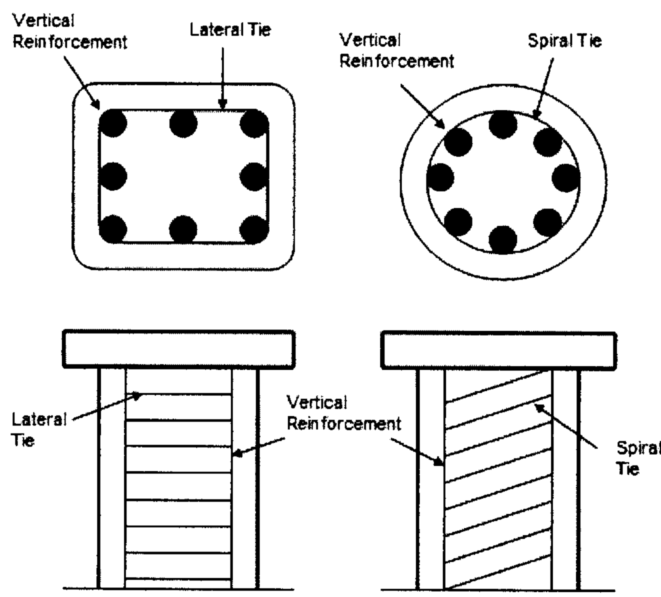


Figure 19: Typical Concrete Column Confinement Detailing

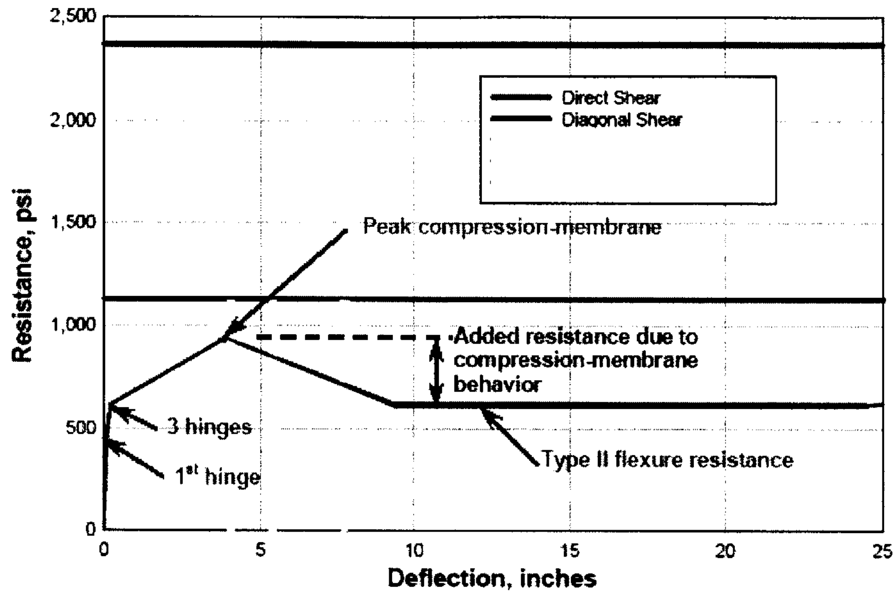


Figure 20: Idealized Concrete Column Resistance Relationship (Malvar et al)

In standard reinforced concrete beams, the main considerations are bending and shear. For most cases, beams need stirrups and longitudinal rebar with special emphasis on the stirrup spacing near joints and tensile reinforcement along the bottom surface of the beam. Such a system would not react well to a load condition that results in uplift forces, load reversals, and torsion. For progressive collapse mitigation, beams should have continuous top and bottom reinforcement and closely spaced transverse reinforcement so that dynamic oscillations may be resisted. Longitudinal reinforcement, especially along the bottom, should extend continuously through joints in order to provide embedment and connection over a damaged area. See Figure 21 which shows a standard reinforced concrete beam that does not have good continuity and subsequent behavior during column loss. As with columns, lap splices should not be placed at mid-span or ends. Mechanical butt splices are recommended so that full capacity is reached between adjacent steel (Smilowitz et al). Seismic design guidelines in chapter 21 of ACI 318-02 should be followed for ductile response in beams. General guidelines on development lengths should be followed. Joints should be designed to be the strongest point of the beam so that shear failure does not occur before formation of plastic hinges and brittle separation is avoided. Monolithic cast beams are preferred because continuous beam behavior results in less deflection and better continuity.

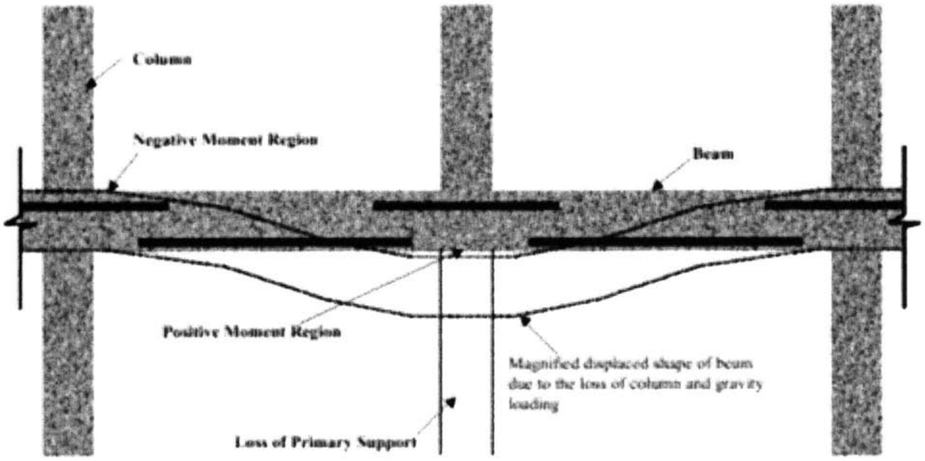


Figure 21: Girder Response from Column Loss

Slab and floor design for reinforced concrete should develop two-way action whenever possible because of the added redundancy and ability to span more than one direction. ACI code requires tensile reinforcement and shrinkage and temperature reinforcement for standard slabs which has been shown to be adequate for resistance of load reversals, but two-way slab design would provide better performance. Similar to beams and columns, seismic detailing is beneficial since it accounts for lateral motion. Top and bottom reinforcement should extend into surrounding beams and columns to ensure embedment and connectivity so that catenary action can occur. Steel provided between adjacent members and the slab also helps to distribute loads if there is an element failure. Punching shear should be considered in the design so that additional loads generated from a column loss can be transferred along to adjacent columns.

Connections are very important for the prevention of progressive collapse and in reinforced concrete structures this depends on the detailing of the reinforcement. The performance of a structure to an abnormal loading is dependent on the strength and ductility of connections. Recently released federal regulations require that reinforced concrete structures be designed with applicable peripheral, internal, vertical, and horizontal ties to columns, beams, and slabs (UFC 2005). These guidelines require the reinforcement to be straight and to have certain design strengths across connections and joints. Continuity of the ties must be provided by either welded splices or Type I or Type II mechanical splices defined in ACI 318-02 (See Figures 23 & 24). Mechanical butt splices tend to achieve better tensile performance than other splice option. Seismic hooks are recommended for anchoring ties in accordance to chapter 21 of ACI. A schematic of the arrangement and locations of the various tie forces that need to be considered is shown in Figure 22.

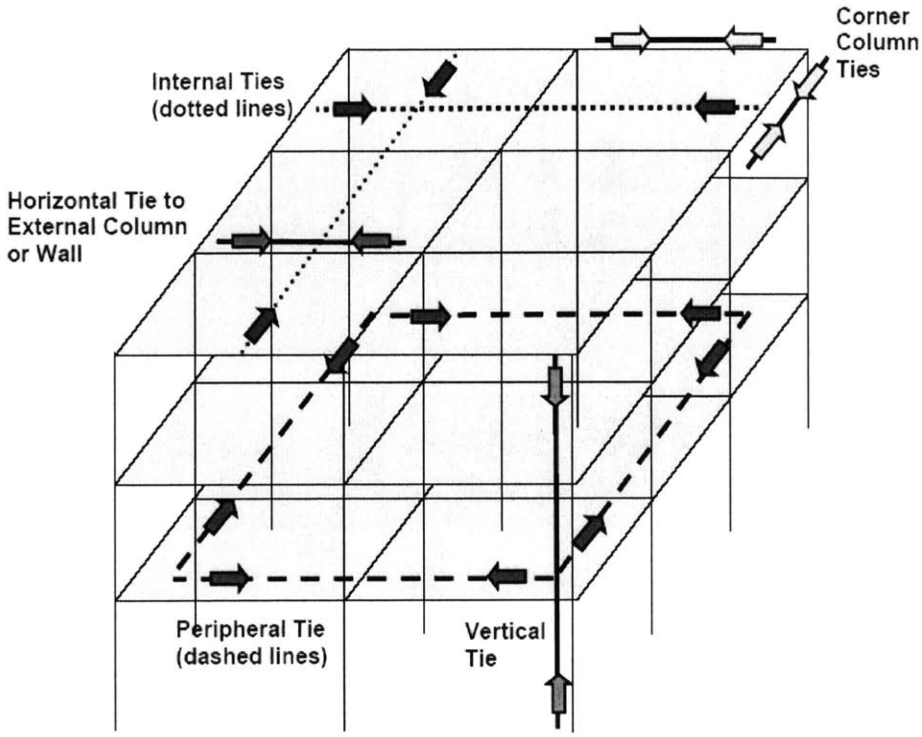


Figure 22: Tie Arrangement (UFC 2005)

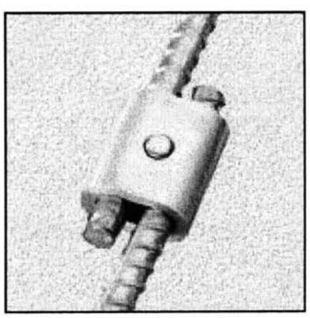


Figure 23: Mechanical Splice Lap (ERICO)

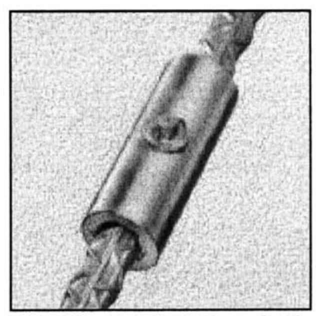


Figure 24: Mechanical Butt Splice (ERICO)

Precast concrete has become popular and attractive in structural engineering due to its speed of construction, but these systems are usually more susceptible to progressive collapse. If precast is used, careful considerations must be made in the design of the reinforcement layout in the panels and connection details. Concrete panels should have reinforcement in two directions closely-spaced to form a mesh that holds the concrete together and allows large displacements. Connections should provide adequate strength to transfer forces to adjacent members and the necessary amount of concrete should be placed over the steel at connections to allow full capacity (Smilowitz et al). All precast members should be analyzed and designed to withstand load reversals. A concrete topping slab can be provided to achieve more continuity and tie members together. It is recommended that the minimum reinforcement ratio for precast panels be 0.25% for thicknesses of 8" or more and 0.5% for thicknesses less than 8" (PCI).

Prestressed and post-tensioned concrete are quite common for long-span girders and parking garages. These systems are designed specifically for certain load carrying scenarios and are not well equipped to withstand abnormal effects. A certain measure of progressive collapse mitigation can be achieved by the addition of continuous top and bottom reinforcement to enhance ductility. It is important to perform accurate analysis on prestressed concrete structures for an abnormal loading to determine the adequacy of the design since brittle failure is a good possibility. The tensioned tendons are under high stress and load reversals can cause abrupt rupture which may initiate a progressive collapse, so these systems should be avoided when there is a chance for extreme loads.

3.2.3 Retrofitting Schemes for Existing Structures

There has been extensive research that is ongoing developing retrofitting schemes for upgrading reinforced concrete systems for progressive collapse. With heightened awareness of terrorist threats and abnormal loads in general, there is a need to retrofit existing structures that are at risk but were designed with little or no consideration of extreme events. The challenge for the designer is performing a thorough site investigation and deciding the most economical way to improve the structural integrity. It is much more difficult to strengthen a built structure because of access to the structural system. Reinforced concrete connections are especially difficult because they are encased in layers of concrete and it is not practical to modify the reinforcement detailing. Therefore the most common retrofitting schemes are external modifications that add strength, ductility, and continuity to the system.

As with any design for progressive collapse, the first step is to determine the hazards and decide to what extent the structure needs to be retrofitted. Next the designer must perform a thorough site evaluation and investigation to determine precisely the details of the structural system. This can be done by reviewing design documentation and to determine whether or not significant structural or architectural alterations were made during the construction process (Dusenberry et al). This may involve on-site inspection with non-destructive testing methods or even physically removing portions of walls to see the actual structural system. The inspection should identify the level of corrosion in the concrete because the material performance is degraded. The investigation should identify critical members and connection details that might be susceptible to progressive collapse.

The most fundamental retrofitting is to prevent the threat from getting to the target by increasing stand-off distances or applying some sort of perimeter defense system. If modifications to the structural elements are found to be impractical and costly, this might be the best alternative. Some possibilities include the use of protective barriers or bollards to resist building impact, or apply energy-absorbing shields to the exterior of the structure. If site constraints or client demands do not warrant these alternatives, the design should identify the most critical members and determine the best way to upgrade to the level necessary based on the risk of progressive collapse.

One of the most effective means to retrofit an existing structure is to add material to the outside of the structural member. This can be achieved through the use of steel jackets, composite wraps made up of Fiber Reinforced Polymers (FRP), and additional layers of reinforced concrete. These methods have been used in the past for seismic retrofits and are good for increasing the strength, ductility, and confinement of members. Of the three methods, composite wraps are preferred because of their ease of application, lightweight, cost, and have the ability to increase the overall strength of the member without increasing the size of the cross-section. Steel jackets can also be effective, but one concern is that they tend to increase the moment capacity such that direct shear failure is possible (Crawford). Typical composites that are used for wraps include Carbon (CFRP), E-Glass (GFRP), and Kevlar (KFRP). The procedure for designing FRP wraps begins with an initial analysis of the non-retrofitted element to some abnormal loading to determine its response. This analysis should be a direct method because it will result in the most accurate and cost effective retrofitting scheme. Flexural capacity should be increased so that maximum allowable displacements are not violated. Next the shear capacity, including specifically diagonal and direct shear, should be calculated and appropriate levels of retrofit must be applied to obtain the necessary confinement and ductility. This process can be used to retrofit concrete beams, walls, and columns quite effectively. Special attention must be paid to the FRP placement around beam-column connections. Research has been done that shows the effectiveness of CFRP to increase ductility and strength of connections in reinforced concrete members (Dusenberry et al). Care must be taken when designing for blast loads because seismic methods for FRP retrofit may result in unfavorable behavior around the column connections. It is recommended for blast design that the FRP extend along the full length of the columns with no gaps at the connection (Malvar et al).

Specific research has been conducted studying the retrofit of reinforced concrete columns with CFRP. This study has developed methods to predict the thickness and type of wrapping retrofit (steel or FRP) that is necessary to produce certain levels of performance based on the column's properties and a computer program has been produced that performs this design procedure (Malvar et al & Crawford et al). In order to justify the accuracy of the design procedure, full-scale tests were run on retrofitted and non-retrofitted columns. A 4-story reinforced concrete building with moment frame connections was constructed using East Coast building code and subjected to blast loads. This type of building is not designed with stringent seismic codes and thus is not expected to display good ductility. One column was non-retrofitted and detailed for inadequate moment capacity and confinement so that the expected failure mode would be a brittle shear mode. The retrofitted column has the same amount of steel, but is reinforced with six horizontal CFRP wraps and three vertical strips on each column face to provide flexural and confinement enhancement as seen in Figure 25 (Crawford et al). CFRP strips were extended through the joint connection region to the next column on the tension and compression sides, with additional wrapping to anchor the strips on the tension side as shown in Figure 26 (Crawford et al). These strips did not suffer any damage and improve the integrity of the connection, but the detailing of the CFRP in these regions should consider which direction the loading may come from so that the connection has equal capacity in all directions.

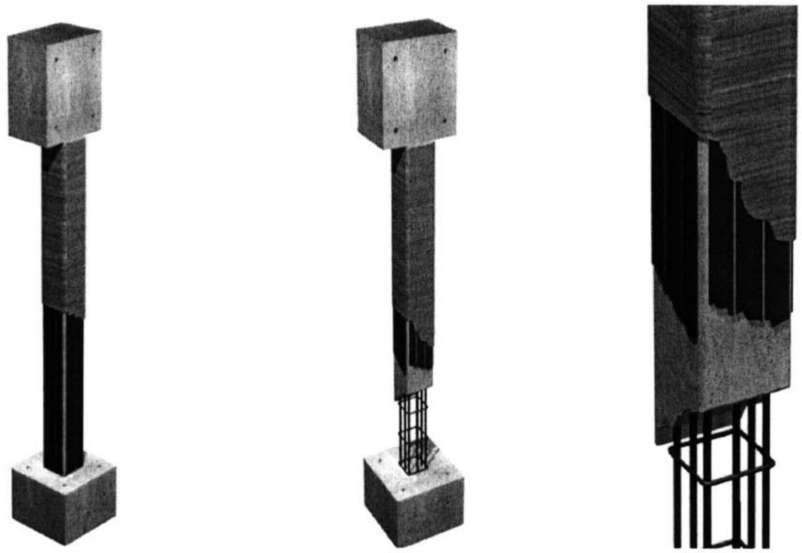


Figure 25: Concrete Retrofit with CFRP (Crawford et al)

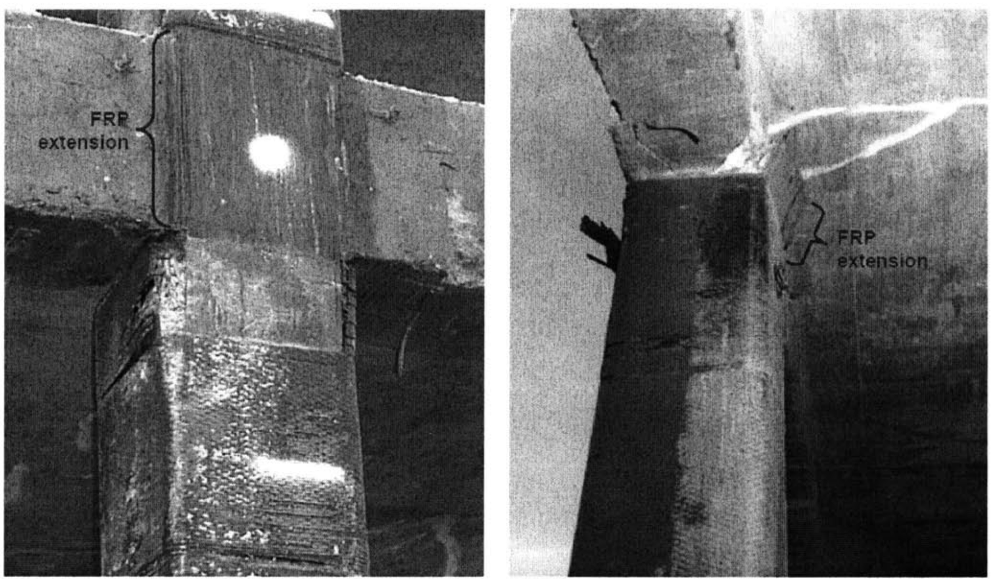


Figure 26: CFRP Details at Joint Interface (Crawford et al)

A close-in blast was detonated near the exterior first floor columns of the building and the response of the two types of columns was evaluated. As was expected, the non-retrofitted column failed in a shear manner near the two ends. It was damaged extensively and experienced approximately 10.5" of displacement at mid-height (Crawford et al). The CFRP retrofitted column faired very well and did not show any damage. The CFRP remained attached and the column did not experience any displacement at mid-height and was able to resist the blast (Crawford et al). See Figure 27 for a comparison of the two columns after the blast.

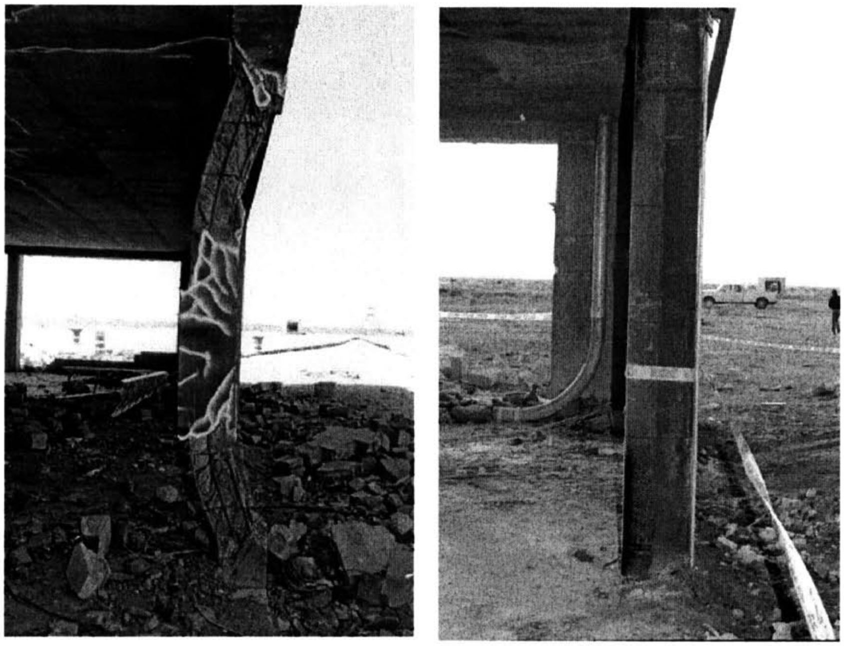


Figure 27: Test Results of Non-Retrofitted and CFRP Retrofitted Columns (Crawford et al)

Bearing wall structural systems have shown to be very resilient against blast loads, but there exists the chance of progressive collapse if portions of the wall are lost which can significantly reduce the integrity of the support system. Retrofitting schemes for bearing walls are aimed towards limiting the possible area of failure by hardening the walls so that they may resist extreme loads with as little damage as possible. Typical retrofits include the addition high-strength paneling that strengthens and reduces deflections in the wall, as well as holding together sections that may become damaged. A few methods that have been used include high strength metal stud walls that provide redundancy, composite panels that stiffen the existing wall, and bonded Kevlar laminate (Crawford). Of these alternatives, Kevlar laminate has shown particularly good response to blast loads and may be the preferred retrofit because of its relatively high strength to thickness ratio and ease of application (See Figures 28 & 29).

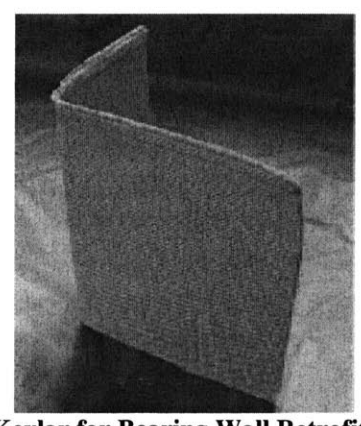


Figure 28: Kevlar for Bearing Wall Retrofit (Crawford)

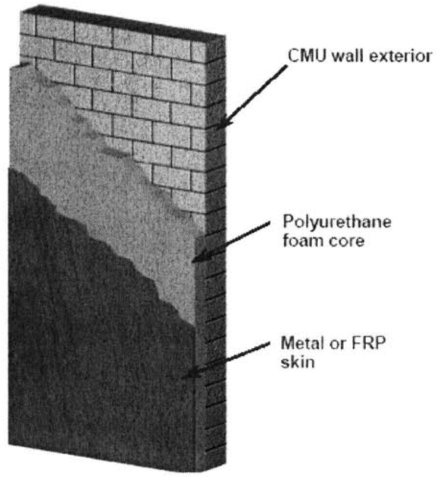
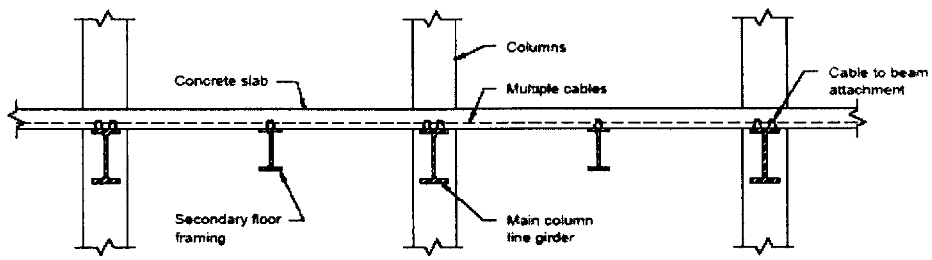


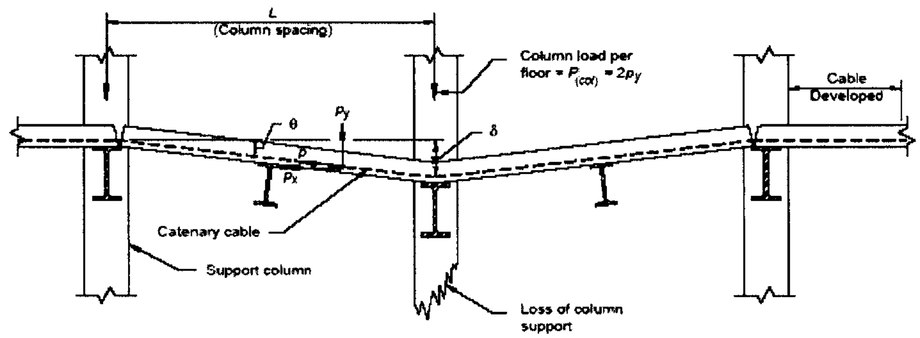
Figure 29: Bearing Wall Retrofit Scheme (Crawford)

Many retrofitting schemes are possible and decisions regarding the correct choice must be thorough and crafted to match the needs of the site. The design recommendations for progressive collapse mitigation can be applied to retrofitting schemes. Although it may not be possible at all times, the designer should strive to add alternative load paths, increase redundancy, and develop two-way action in structural members. Two-way action is achieved in reinforced concrete primarily through the steel reinforcement detailing, and thus is difficult to alter as a retrofit. One alternative is the addition of new beams to provide more support and new load paths. Another option is the addition of diagonal members to create truss action, or upgrading the lateral support system to develop vierendeel truss action which adds to the overall robustness (Dusenberry et al). A recent development is the implementation of strong floors distributed throughout the building that are strengthened to resist progressive collapse of a certain amount of weight from above. Studies conducted by Simpson Gumpertz and Heger Consulting Engineers found that a strengthened floor can support as many as 10 floors (Carter & Shipe).

Precast and prestressed concrete systems are usually more prone to progressive collapse due to their weak connections and poor detailing. One retrofitting method that has shown the ability to increase ductility in response to abnormal loading conditions is the use of external cables (Crawford). This scheme involves tying precast panels or prestressed girders together to columns with externally attached cables. In the event of a column loss, the panels or girders sag down onto the cables and remain secured due to catenary action and loads may be redistributed across to nearby columns. The Khobar Towers had this system which is credited with limiting the extent of progressive collapse to only half of the building. Connection details for the cables are important so that they remain attached after column loss and a typical connection is shown in Figure 30.



(a) Undeflected shape.



(b) Deflected shape.

Figure 30: Cabling Retrofit Scheme (Crawford)

3.3 DESIGN ISSUES WITH STEEL STRUCTURES

Steel is a very robust material that has many favorable properties that allow it to be used in a variety of applications. It is one of the top two structural materials used throughout world and thus its behavior in response to abnormal loads must be examined to determine how steel structures can be composed to resist progressive collapse. With steel being less massive and generally slender, there are concerns about its behavior under large dynamic forces. There are some advantages of steel structures, but the weaknesses may outweigh any perceived advantage over reinforced concrete for progressive collapse mitigation. Nonetheless, designers and researchers have studied steel behavior and determined ways to improve redundancy, ductility, and overall structural integrity. These recommendations will be presented for both new construction and retrofitting existing structures.

3.3.1 Strengths & Weaknesses

The material properties of steel are well-suited for structural engineering due to high overall strength, high modulus of elasticity, and good ductility allowing it to undergo large inelastic strains prior to ultimate failure. Another favorable feature is that it is an isotropic material with essentially equal strength in compression and tension and thus does not need additional reinforcement to acquire strength for specific loading conditions. Unlike concrete, a symmetric steel section will offer the same resistance to gravity loads as it would for an opposite uplifting force, a scenario that is likely to be encountered from blast loads. The ductility of steel is very good and generally additional reinforcement or confinement is not required for steel sections, thus making it easier to obtain a progressive collapse resistant design. Steel sections tend to be smaller than concrete due to their high strength to weight ratio which results in a lighter overall structure. A reduced weight means that a local failure will not induce as much redistributed forces to surrounding members, thereby increasing their chance of halting the collapse. Steel is a very dense material and thus great strength can be achieved at a minimum thickness of material. Steel structures are generally lighter, but there are times when thick sections are used that can be quite heavy, such as a heavy tank or battleship which are designed to have great blast resistance.

Whenever there is an associated risk for progressive collapse in steel structures special attention needs to be placed on the integrity of the connections. There is a general lack of member continuity throughout steel structures due to the termination of members at all joints. Load transfer and element connectivity are dependent on the performance of the connections. Unlike in reinforced concrete, it is difficult to achieve catenary action in beams because standard connections simply join beam ends to the column, and thus a column failure would also result in separation between the adjacent beams. Dynamic blast loads have been shown to cause brittle crack mechanisms at welded joints that prevents the development of full member capacity (Houghton & Karns). Achieving adequate joint strength and ductility is very difficult due to the complex nature of stress distributions. As a consequence of its lightweight, steel structures lack the mass to resist the dynamic effects of abnormal loads and are more likely to be excited by the high inertial forces. This means that the structure could experience modal deflections such as sway or twist. Steel does not have very good thermal resistance and thus fire-proofing must be provided. Since steel members have relatively small cross-sections, the response of the façade or cladding will be more important to the structure's behavior. A typical failure scenario is for the cladding and floor system to be removed, leaving the framing intact because the steel members can not provide much area of resistance against the blast load (Smilowitz et al). Alternatively, if the cladding is able to remain intact, it will induce larger loads to the columns because the loaded tributary area is greater (Houghton & Karns). There are many concerns with steel systems that increase the susceptibility to progressive collapse and thus care must be taken to develop proper detailing.

3.3.2 Suggestions for New Design

An effective structural design for prevention of progressive collapse using steel involves a variety of considerations that should be analyzed and evaluated. As always, a thorough inspection of the site and perimeter defense mechanism should be implemented. Many of the same perimeter options detailed in previous sections are applicable to steel structures, such as restricted access and protective barriers. Arguably the most important aspect of the design is the connections. Due to the nature of steel connections, bolted or welded, there is a high probability of failure at the connections before ultimate capacity of the members can be obtained. This poses a major threat of progressive collapse because there is a general lack of continuity across connections in steel structures. The selection of steel sections for girders and columns needs special attention so that appropriate levels of flexure, shear, and torsion resistance are adequate for the desired performance. Buckling is more important in steel than concrete due to the slenderness of the sections and stiffeners should be used where appropriate. Floor slabs typically are composite metal decks with concrete slabs which need special detailing in the face of abnormal loading conditions. The structural framing system needs adequate redundancy and alternate load paths and it is recommended that moment or braced frame be used.

The design of columns for progressive collapse considerations must contain analysis for buckling, capacity for resisting flexure, shear, and torsion, and the column should have higher strength than the adjacent beams. A general rule of thumb is to select columns that have low slenderness ratios in that this ensures its axial capacity and alleviates buckling concerns. Columns should display ductile behavior which is usually controlled by the connections since steel has excellent ductile material properties. The column should be designed for adequate flexural capacity based on a design abnormal load or to carry additional loads that must be redistributed from a lost column. Shear capacity must be checked so that it is greater than the flexural capacity so that a ductile failure mode controls. Torsion is a concern for open steel sections which may experience twist due to blast loads. Therefore it may be beneficial to select closed shapes such as HSS members which have good torsional resistance and also provide equal flexural capacity in all directions (Longinow & Alfawakhiri). Once the column strength is assured, additional checks for buckling must follow. A check of column stability for the loss of adjacent beams and resulting increased unbraced length must be conducted. Stability can be enhanced by the use of lateral flange bracing or encasing steel with concrete following the AISC Seismic Provisions (Smilowitz et al). Column stiffener plates as well as web doubler plates are also effective for limiting local buckling due to abnormal loading response (Smilowitz et al). It is also important that the design consider the tributary area of the load being applied to the column through the façade. The magnitude of the load felt by the column is dependent on whether the façade remains in tact or fails, thus the design must consider the worst case scenario.

The biggest concern with steel beams and girders is local buckling and behavior at connections. Load reversals are not as critical for steel because the material properties allow for equal strength in any direction, depending on the cross-sectional shape. For instance, W-shape girders will have the same flexural capacity for gravity loads as it will for uplift as long as the connections are detailed correctly. Beams that support concrete slabs should employ shear studs to ensure adequate connection (Smilowitz et al). Lateral torsional buckling is a concern because it can weaken a beam and lead to failure, thus adequate stiffeners and bracing must be provided. Lateral support should be provided along the full length of the beam so that plastic hinging can occur for ductile response. Connections should allow development of plastic hinge and beam axial tension capacity so that brittle failure is avoided and catenary action may occur if a column is lost (Smilowitz et al). A good reference for detailing connections is the AISC Seismic Provisions for Structural Steel Buildings, and a more detailed discussion on connection recommendations will follow in the next few sections.

Slab design for steel structures is usually a composite system with a concrete deck supported by steel beams. As was previously mentioned, shear studs should be used to obtain good bond and composite action between the concrete and steel. Use of concrete slabs for steel systems is good practice because it serves as a continuous lateral brace for the steel beams and girders. The remaining design guidelines should follow those given for reinforced concrete slabs. Detailing should include two-way reinforcement and continuous rebar across joints to provide better redundancy and ductility in the slab system.

The design of connections between beams, columns, and slabs needs special attention to resist progressive collapse. All of the design recommendations given thus far for steel members are only as good as the design of the connections. The reason is that the connection is the weak point of the structural system which must be able to transfer loads across discontinuities which typically experience complicated stress patterns. Frame capacity for steel structures is governed by member response and connection response (Crawford). Member response is well understood while connection response is not, especially when complicated by a blast loading scenario. What is known is that connections should allow for full flexural capacity of attached members, thus allowing plastic hinges to form. Obtaining such response from traditional welds or bolted connections is difficult, and there exists some doubt as to whether sufficient performance for progressive collapse situations can be reached. Some researchers have recommended use of full-penetration fillet welds for toughness, or high strength bolts (Smilowitz et al). Others note the deficiencies of traditional connections, even those specified for seismic conditions due to uncertainties in the weld quality and the chance that brittle fracture can occur (Houghton & Karns). Traditional connections are typically described by a T-joint configuration groove weld that joins the girder flanges directly to the face of the column flange, or bolts may be used to provide the connection across the column (Houghton & Karns). This connection inherently does not provide continuity across the joint for the girders such that if column loss occurs, there is a good chance that the connection will be lost as well. This brings up an important flaw in steel connections because there is no way to ensure girder-to-girder connection across the column without using the column in traditional connections. Even when columns do not fail, connections may fail due to brittle cracking which can lead to rupture of the entire flange connection. The reasons for this type of failure are due to complex tri-axial strain concentrations at the weld resulting from tensile, flexural, and shear strains, as well as localized lateral torsional buckling of the flange (Houghton & Karns). See Figure 31 for examples of brittle failures in traditional connections. This complex concentration of strains results from the various internal forces created by an abnormal load that bunch up at a joint. This behavior is extremely complicated and difficult to model, making connection design arduous and rather unreasonable to prevent by traditional methods.

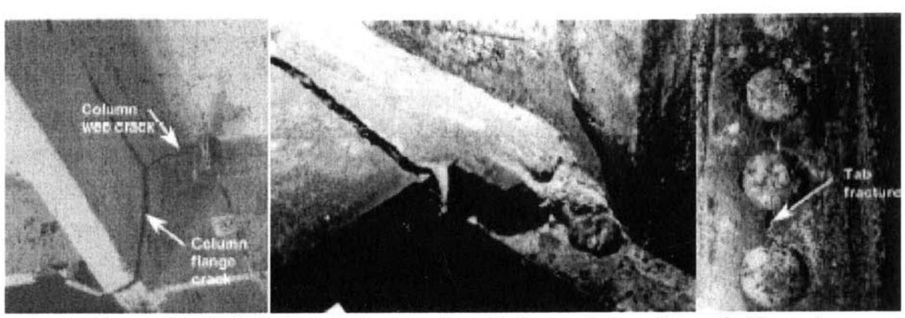


Figure 31: Brittle Steel Connection Failures (Houghton & Karns)

There is a new type of connection that has been developed that appears to provide the needed strength, ductility, and member continuity across joints called SidePlate™. This steel connection, pictured in Figure 32, is composed of two parallel side plates fully-welded to plates connected to the girder flanges, thereby separating the column face and the ends of the girders. This separation eliminates the highly restrained condition and reduces the tri-axial strain concentrations (Houghton & Karns). In addition this connection provides torsional rigidity due to its box like configuration around the girder ends and column, which is said to assist in resisting local flange buckling and dynamic shear loads (Houghton & Karns). Another key feature is the continuity this system provides for girder-to-girder connection through the side plates, therefore if the column is lost, the girders remain attached and catenary action may occur. In order for the SidePlate™ connection to work efficiently, the parallel side plates must be specified for greater strength than the girders so that plastic hinges may form away from the connection. As with any welded connection, quality assurance is a concern, but all SidePlate™ connections are welded in the shop and come to the site in column tree sections. The four girder splice options for these prefabricated connections are fully-welded CJP butt joint, bolted matching end plates, fillet-welded flange plates, and bolted flange plates (Houghton & Karns). The SidePlate™ technology has been used successfully in several blast-resistant design applications and detailed computer simulations have been performed that prove its effectiveness. Nonlinear three-dimensional models were developed for traditional and SidePlate™ connections that were used in a simulation following the GSA's missing column analysis. The results showed a marked reduction in stresses around the column and indicate redundancy through the girder-to-girder link, which can be seen in the following Figure 33 (Houghton & Karns). Note in Figure 33 that the SidePlate™ connection shown on the bottom has the stresses shifted away from the girder ends as opposed to the traditional connection.

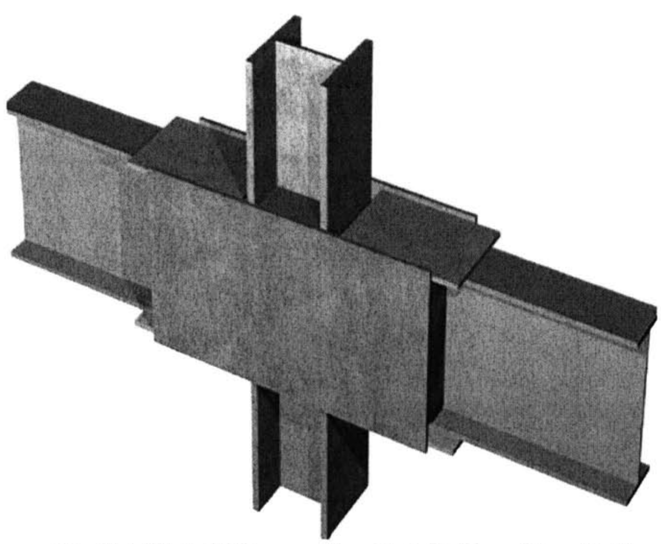


Figure 32: SidePlate™ Connection Detail (Houghton & Karns)

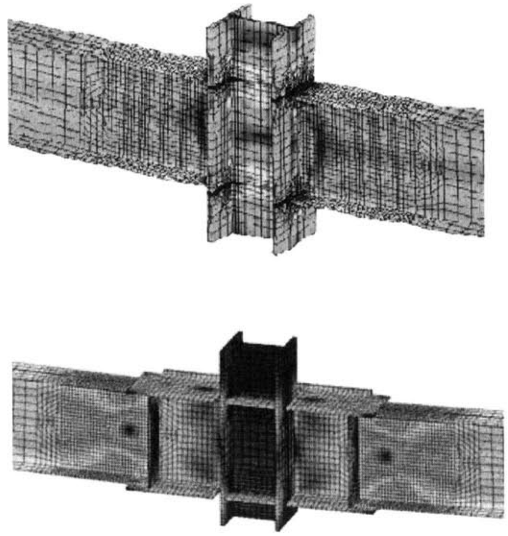


Figure 33: Comparison of Stress Concentrations of Connections (Houghton & Karns)

One area in which steel contains several design options is redundancy and introducing additional load paths to the system. Redundancy can be designed for by the addition of diagonal members of "X" bracing to steel frames such as shown in Figure 34. Diagonal members could be applied to each floor individually or placed externally over several floors which may also serve a dual purpose as part of the lateral bracing system for high-rise buildings. SidePlate™ connections come in several braced frame options which may create "X" brace patterns and enhance redundancy, ductility, and stiffness (Houghton & Karns). Another consideration is the use of a strong floor or megatruss. This may be more applicable to high-rise buildings, but the main idea is to position strengthened floors at various locations such that they are designed to restrain a progressive collapse of the floors from above or below. For instance, a tall building with strategically positioned hat trusses could be able to survive a local failure at the strengthened floor.

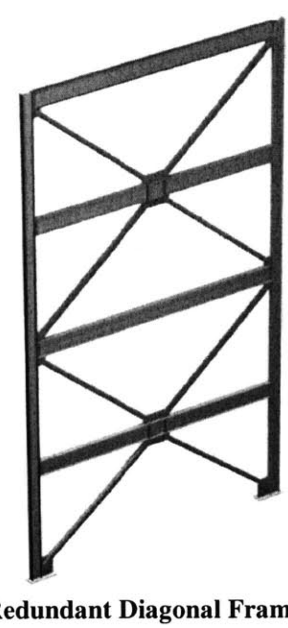


Figure 34: Redundant Diagonal Frame (Crawford)

Various design suggestions have been presented for enhancing progressive failure resistance for steel structures. It should be stressed that understanding the complex material-level mechanics of connection behavior is essential in order to produce an appropriate design. The SidePlate™ design shows great promise as a new connection detail that appears to provide the best joint behavior for progressive collapse mitigation. Its use has been studied and verified through several successful blast-resistant designs including an air traffic control tower, a U.S. courthouse, and a naval command headquarters (See Figure 35).

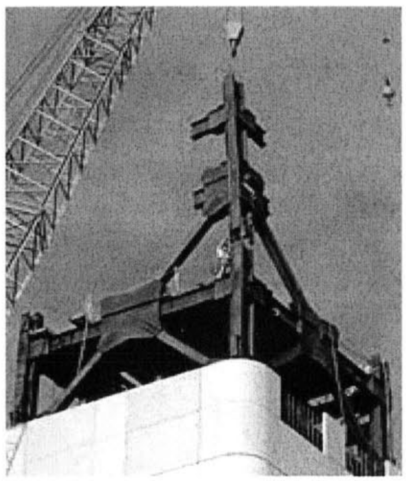


Figure 35: SidePlate™ Application (Houghton & Karns)

3.3.3 Retrofitting Schemes for Existing Structures

The retrofit of already built steel structures is important when progressive collapse resistance in substandard. The process of designing a retrofit must be thorough and include an extensive investigation of the structure's support system and detailing so that problem areas may be identified for strengthening. Special attention must be placed on the connection specifics so that the structure's behavior can be understood under an abnormal loading. Corrosion should be checked, but most structures such as buildings have cladding protecting the steel from corrosive elements and shouldn't experience much deterioration. As with concrete retrofitting, standoff distances should be increased and perimeter defense measures should be implemented. These initial steps help to produce an efficient retrofitting scheme because it allows the designer to focus on the critical elements. This is important because retrofitting is not an easy procedure due to site constraints and accessibility, therefore the designer will need a scheme that will result in a minimum of disruption to the facility.

One retrofitting scheme is to strengthen key members that have a high risk of extreme loading such as first-floor columns. Composite wraps and jacketing techniques are not very effective with steel members. A more effective method is encasing or filling steel columns with concrete which adds strength, toughness, and resilience to the member (See Figure 36). Another retrofit alternative is to add steel plates to the exterior of the column, thereby increasing the thickness of the section at certain places to get a required response, such as increased flexural capacity. Such retrofits must check that the increased strength does not cause a brittle shear failure mode to control the column response.

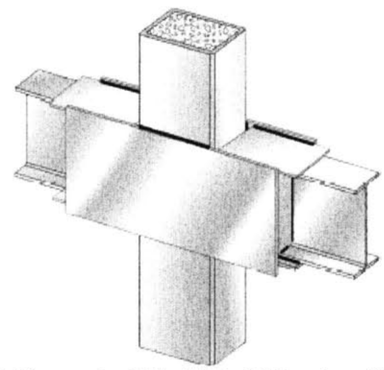


Figure 36: Concrete-Filled Steel Member (Crawford)

Since steel connections are so important, it is most likely that the most effective retrofit will entail some sort of connection upgrade. Retrofitting existing connections is difficult due to accessibility. One connection upgrade is the addition of cover plates over the existing girder-column connection to increase the joints capacity (Dusenberry et al). A better alternative would be the use of the SidePlate™ connection that provides better capacity, ductility, and redundancy. The replacement of existing connections with SidePlate™ involves removing the existing joint penetration weld by air arcing and welding the side plates with fillet welds loaded principally in shear (Crawford). The layout of the side plates requires the need for an access window to be opened in the plate so that the welding can occur inside, which is then closed once the welding is complete.

When redundancy and alternate load paths are required, there exist several options for retrofit. One is to introduce new members in the structural framing that provide additional load paths, such as diagonal braces. Another alternative is the addition of strong floors or megatrusses. Both of these concepts share the same general goal of positioning strengthened locations throughout the structure so that they may contain a local failure from becoming a progressive collapse. A good lateral bracing system may assist the structure to span over a location of damage by vierendeel truss action (Dusenberry et al). Such a retrofit though is in most cases unpractical due to the magnitude of the retrofit that would be required. The use of cables has been discussed with the principle of hanging the structure by engaging tensile forces in members so that catenary action occurs after a local failure (Dusenberry et al). In some cases this has been accomplished by draping cables along column lines for added resiliency. Cable tensile capacity must be checked to ensure that redistributed loads do not cause an overloading and subsequent cable snap.

4. CONCLUSIONS

It is becoming more evident in the realm of structural engineering that there is a need to recognize abnormal or extreme loading events that can compromise the structural integrity of structures and lead to progressive collapse. Measures should be taken to avoid this type of failure because of the level of destruction, damage, injury, and death that may result. The current state of affairs in society is one that is conducive for real threats and targeted attacks to can produce a progressive collapse type event. Designers should be aware of the threats to their structures and strive to make them as safe as possible. There is the issue of liability and the uncertainty with abnormal loading events. It is very difficult to assess what exactly the threat is and how large of an impact it will have, and there may be cases where it would be impossible for the designer to have developed a structure with sufficient strength. Currently there are very few official codes or standards for this type of design in large part due to the random nature of these events and a lack of widespread support for the need and validity of design guidelines for abnormal events. What is becoming clear is that there is indeed a frightening trend towards increased terrorist action aimed at imparting destruction on structures that serve as political and cultural targets. It would be ignorant for the engineering community to remain content with traditional building codes and standards and allow economics and a lack of knowledge about abnormal loading to become excuses.

The only way that extreme events and progressive collapse mitigation measures to become standardized is for a motivated effort from owners, academia, engineering professionals, and government agencies to come to the realization that there is need and to push for continuing research to develop better tools and design paradigms for structures under abnormal conditions. There have been positive efforts in recent years that arose from highly publicized structural collapse events, but it is important that these efforts are continually supported and not forgotten as time goes by. Arguably the most important aspect to develop for standardization is a well-defined approach of identifying the threat probabilities and relating that to design loads or deformation criteria. Better analytic simulations and software are needed to obtain more accurate structural responses to abnormal loading events so that the resulting behavior can be well understood. A better overall understanding will lead to more accurate design specifications and response criteria. Complicated computer models need to be developed and verified against real data obtained from full-scale tests. Also, new retrofitting and performance enhancing structural techniques should be encouraged and supported that pertain to progressive collapse mitigation.

There are signs that progressive collapse is becoming a topic of increased awareness after the World Trade Center collapse, especially with the GSA development of guidelines. Still this publication is by no means a perfect solution and needs more information and extension to a wider range of applications. This county can look at building codes developed abroad that contain specific clauses for the prevention of progressive collapse based on extreme events. Countries such as Canada and Great Britain have explicit statements and criteria for structural integrity for extreme loading. It is interesting to note that these statements have been changed and modified many times, perhaps for reasons including economics, criteria set too high, or ineffectiveness. These past experiences should be examined to develop more widespread regulations in the U.S. It should be noted that abnormal loading and progressive collapse is contained in some codes, but usually consists of a few vague statements that require the designer to ensure the structure's overall integrity. These statements are at the discretion and interpretation of the engineer, thus the implementation will depend on the knowledge of the engineer, client, type of project, and many other factors.

One topic that hinders the development of better codes and standards is the liability of the engineer or designer once action is taken to achieve a progressive collapse resistant structure. It is highly unlikely that any code or standard will be able to develop designs that are will be able to consider each possible extreme event and resist all possible failure modes. Thus developers must choose their language carefully and make it clear that the guidelines and suggestions are adequate only up to the level of knowledge and certainty available for preventing catastrophic failures. Much knowledge is already available, and with following the main principles of redundancy, ductility, and structural integrity structures can be made to behave well and resist abnormal loads at a reasonable level. Combining the knowledge available and the suggestions given in this report will be difficult to produce into universal code, but it is possible with the right efforts. It is clear that there is a renewed effort among engineers, academia, and government officials in the wake of 9/11 to identify the reasons for progressive collapse and implement better detailing so that structures may behave in robust and ductile manners. Research and efforts among the structural engineering profession have dealt with progressive collapse since the late 1960's after the Ronan Point collapse, and considerable information has been gathered. The understanding of collapse mechanism and prevention is relatively well known, but a better understanding can still be achieved. Even so, the knowledge available at the current time is capable of mitigating progressive collapse and these methods should be implemented in structures where abnormal loads are a real threat.

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