

**Engineering and Organizational Issues Related to the
World Trade Center Terrorist Attack**

Volume 4

**From the WTC Tragedy to the Development of Disaster
Engineering for Landmark Buildings – An Extension of the
Performance-based Earthquake Engineering Approach**



By George C. Lee, Vladimir Rzhevsky, Mai Tong and Suwen Chen



▲ *The Multidisciplinary Center for Earthquake Engineering Research*

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the University at Buffalo, State University of New York, the Center was originally established by the National Science Foundation (NSF) in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

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Engineering and Organizational Issues Related to the World Trade Center Terrorist Attack

Volume 4: From the WTC Tragedy to the Development of Disaster Engineering for Landmark Buildings - An Extension of the Performance-based Earthquake Engineering Approach

By George C. Lee, Vladimir Rzhevsky, Mai Tong and Suwen Chen
Multidisciplinary Center for Earthquake Engineering Research
University at Buffalo, State University of New York
Buffalo, NY 14261

and the

Department of Civil, Structural and Environmental Engineering
University at Buffalo, State University of New York
Buffalo, NY 14260

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MCEER-03-SP04
Red Jacket Quadrangle
Tel: (716) 645-3391; Fax: (716) 645-3399; Email: *mceer@mceermail.buffalo.edu*
World Wide Web: *http://mceer.buffalo.edu*

Foreword

The terrorist attack that took place on September 11, 2001 in New York City resulted in thousands of lives lost, the collapse of the twin towers of the World Trade Center as well as damage to adjacent buildings, and extensive disruption of transportation and other lifeline systems, economic activity, and other social activities within the city and the surrounding area. When the final accounting takes place, this attack will almost certainly constitute one of the most deadly and costly disaster events in U. S. history.

In a very real sense, the September 11 tragedy, the nature of the damage that occurred, the challenges that the city's emergency response faced, and the actions that were undertaken to meet those demands can be seen as a "proxy"-albeit a geographically concentrated one-for what a major earthquake can do in a complex, densely-populated modern urban environment. Like an earthquake, the terrorist attack occurred with virtually no warning. As would be expected in an earthquake, fires broke out and multiple structural collapses occurred. As has been observed in major urban earthquakes and in other disasters (e.g., Hurricane Andrew), structures housing facilities that perform critical emergency functions were destroyed, heavily damaged, or evacuated for life-safety reasons. Additionally, because the majority of the damage occurred to relatively new and well-engineered structures and because the emergency response system in New York City was considered very well prepared for all types of emergencies, particularly terrorist attacks, the attack and its aftermath provide a useful laboratory for exploring a variety of engineering and emergency management issues.

In this perspective, the Multidisciplinary Center for Earthquake Engineering Research initiated a research project (funded by the National Science Foundation) to collect perishable data in the aftermath of the attack for later study to gain a better understanding of how resilience is achieved in both physical, engineered systems and in organizational systems. The project is divided into two major components, focusing on the impact of the disaster on engineering and organizational systems:

- (a) Damage to Buildings in the Vicinity of Ground Zero - The objective of this effort is to collect perishable information on the various types of damage suffered by buildings at Ground Zero, including, most importantly, those that suffered moderate damage from the impact of large debris but that

did not collapse, and to investigate whether state-of-practice analytical methods used in earthquake engineering can be used to explain the observed structural behavior.

(b) Organizational and Community Resilience in the World Trade Center Disaster - The objective of this effort is to collect information on the response activities of the City's Emergency Operations Center and on other critical emergency response facilities. Of particular interest is to identify the plans that were in place at the time of the disaster, as well as how decision systems and remote sensing technologies were used and coordinated with engineering decisions. Efforts will also include identifying the technologies and tools that were most useful or failed (or did not meet expectations) during the emergency period, the types of adaptations that had to be made by these organizations, how well intra-organizational communication and coordination functioned, and whether any emerging technologies were used during the emergency period.

The MCEER special report series "Engineering and Organizational Issues Related to The World Trade Center Terrorist Attack" was initiated to present the findings from this work. The decision to publish a number of brief individual reports focusing on different topics was prompted by the desire to provide timely access to this information. As such, each report in the series focuses on a narrow aspect of the disaster as studied by MCEER researchers. A compendium of these short reports is planned at a later time. It is hoped that this work will provide a useful contribution that can lead to a better understanding of how to cost-effectively enhance the resilience of buildings against catastrophic events.

Dedication

We dedicate this special report to the victims of the WTC disaster. On the eve of the second anniversary, special support and thoughts go to the loved ones of the victims. This tragic event has significantly increased the public's awareness of the need for and importance of disaster mitigation and crisis response. It calls for the research community to speed up its pursuit of cost-effective solutions and countermeasures to mitigate and reduce the impact of extreme multi-hazard attacks on our society.

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1.0 Introduction

The September 11, 2001 (9/11) terrorist attack on the World Trade Center (WTC) complex produced an unprecedented disaster in New York City and the United States. Many technical studies and investigations are continuing to pursue various detailed aspects of the disaster's impact. From a strictly structural mechanics' point of view, the collapse of the WTC towers was mainly the result of extreme multi-hazard loadings (collision, fire and explosion) that subjected the buildings to forces that were beyond the hazard capacity that they were designed to withstand.

In this report, the authors first review the various hazard loadings that were applied to the WTC towers (impact of airplanes, fire and explosion), mainly reported by others, to identify potential multi-hazard design improvements for structural members and their connections, as well as for entire building systems.

There are many specific lessons that can be learned to improve structural design (e.g., redundancy, alternate load paths, strong connection design, building geometric configuration, fire proofing, advanced construction materials, etc.) from this disaster. These lessons are examined from the viewpoint of preventing or reducing the possibility of similar disasters causing the same amount of destruction in the future. Useful information can be applied to future design of buildings and their components to mitigate the impacts of various hazard-loading conditions.

The airplane impact, the fire and subsequent explosion all gave clear indications that the buildings were subjected to multi-hazard loading conditions until they collapsed. The authors believe that a significant lesson learned from the September 11 disaster relates to the need for a new approach to plan, design and construct signature buildings that may or may not be subjected to similar conditions where multiple hazards, and their resulting loading conditions, occur simultaneously. Cost-effective designs that consider all hazard mitigation measures (even for those hazards that are not going to happen at the same time) are needed for such buildings.

The authors have taken a new perspective in this study, proposing a new building performance requirement related to a major, unplanned disaster referred to as "delayed building collapse." The idea is targeted at major disasters where the normal design requirements used by professional practice (architecture, structural engineering, construction, etc.) have been far exceeded and that the only important priority is to "save more lives" when

disaster occurs. With these considerations as a focal point, the performance-based earthquake engineering approach is extended to formulate a framework for multi-hazard engineering activities.

In essence, this has been the basic research strategy of the Multidisciplinary Center for Earthquake Engineering Research since 1997, where earthquake hazard mitigation efforts are considered together with emergency response and recovery activities in dense population centers, particularly those unprepared for possible destructive earthquakes. This vision is extended in the study of the WTC disaster to develop new guidelines for planning, designing and constructing multi-hazard resilient buildings.

1.1 Background of the WTC Attack

On September 11, 2001, at 8:45 a.m., terrorists hijacked an American Airlines Boeing 767 (flight 11), purposely steering it into the North Tower (WTC1) of the twin 110-story buildings that comprised the World Trade Center. The plane, flying southward, crashed into the north side of the tower, penetrating the building at the mid-width of the façade and cutting 31 to 36 of the 59 columns (FEMA, 2002) along that façade of the building from the 94th to the 99th floors. Eighteen minutes later, a second hijacked Boeing 767 flying northward, crashed into the South Tower (WTC2), impacting the eastern third of the south façade of the tower, badly damaging the building shell from the 78th to the 84th floors. Despite the gross damage caused by the penetrating planes, the towers withstood the pounding force of the collision without immediate collapse.

In both cases, fuel from the jets ignited at impact. Debris from the planes and the buildings were ejected into the adjacent space and buildings, principally in the directions of impact. The fuel-fed fires raged at the stories where the collisions occurred, spreading rapidly to the stories above.

At 9:50 a.m., WTC2 collapsed. As the segment of the tower above the impact area started to tilt eastward, almost like a rigid body, it penetrated the stories below, and pushed the building façade outward, which led to a progressive collapse that demolished all the remaining stories below. While the collapse of WTC2 took only between 9 to 15 seconds, according to news reports, the collapse itself did not begin until 56 minutes after the initial impact. At 10:28 a.m., WTC1 also collapsed, presumably triggered by a different combination of collapse mechanisms. Based on videos (shown by the media in the aftermath of the collapse), the stories above the impact areas of WTC1 started falling straight down on the stories below while pushing the façade outward, similar to the fall of the WTC2. It took one hour and forty-two minutes after the impact before the progressive collapse of the WTC1 began.

Relevant information about the design and construction of the WTC towers is briefly reviewed in Appendix A.

1.2 Report Objectives

This report is the fourth in a series that summarizes studies at MCEER following the WTC attacks. The first report documented damage to the WTC complex and the area immediately surrounding it in the days following the attack, while the second report examined the applicability of earthquake engineering models to explain the damage and redundancy in the nearby Liberty Building. The third report investigated the application of a variety of remote sensing technologies for use in emergency response and recovery following a major disaster such as the WTC collapse, a major earthquake, etc. This report begins with a review of the major hazards (collision, fire and explosion) that were factors in the collapse of the WTC towers. It then advances a performance-based engineering approach, referred to as "multi-hazard engineering," that combines knowledge accumulated in earthquake engineering design, hazard mitigation methods and structural response control approaches with lessons learned from the WTC collapse. A new performance level category, "Catastrophe limitation" is proposed in addition to the limited safety and catastrophe prevention categories, which is intended to delay the inevitable collapse of a building to allow occupants enough time to safely evacuate.

The purpose of this report is not to determine the extent and causality of these hazards to the collapse of the towers, but instead to use this knowledge to formulate a rational and reasonable multi-hazard design platform. Further, several assumptions had to be made to carry out calculations that were in keeping with the probable cause of the collapse of the WTC towers. These were made using the authors' best judgment when specific information was not available. Specific details contributing to the actual collapse of the WTC towers are left to others currently involved in rigorous studies of these hazards and their resulting collapse. Therefore, the conclusions obtained in this study may differ from those obtained through study of the actual buildings. In this report, the collision, fire and explosion hazards, and the structural behavior and design of the WTC towers, are addressed with the following emphases:

1. Understand the magnitude and scope of the extreme loadings applied to the buildings caused by the plane crash, fire and explosion.
2. Review relevant information (technical papers and reports, visual and descriptive materials from the media, as well as other information

available to the general public) describing the structural responses of the WTC towers.

3. Summarize the time-dependent responses of the buildings under multiple hazard loading including damage initiation and subsequent collapse of the buildings.
4. Based on the performance-based earthquake engineering approach, framework a new design perspective on multi-hazard performance objectives for the design of disaster resilient buildings.

Finally, the newly defined “multi-hazard engineering” is meant to emphasize an integrated and cost-effective disaster operation against all types of serious hazards. It includes information development, coordinated efforts in mitigation, and emergency response and restoration, with a focus on saving the lives of occupants when structural collapse is imminent. The proposed framework must be complemented by a parallel effort to educate a new generation of engineering professionals who can effectively carry out research and development, implement policies and technical approaches, as well as plan, design, construct, manage and maintain landmark buildings in dense population centers.

2.0 Collision

The WTC tragedy began when the airplanes collided into the two towers. As reported in FEMA (2002), the estimated velocity of American Airlines Flight 11 at the moment of collision into WTC1 was 470 mph, and, likewise, United Airlines Flight 175 collided into WTC2 at 590 mph. Both airplanes were estimated to be carrying 10,000 – 20,000 gallons (38,000 – 76,000 liters) of fuel.

Relevant information about the Boeing 767 aircraft related to the impact is as follows:

Fuel capacity	23,980 gallons (90,770 liters)
Cruise speed	530 mph (850 km/h)
Engines	Pratt & Whitney PW4062/GE CF6-80C2B7F
Overall Length	159 feet 2 inches (48.5 meters)
Vertical Projection	52 feet (15.8 meters)
Maximum Weight	395,000 pounds (179,170 kilograms)
Maximum range	6,600 nautical miles (12,200 kilometers)

According to FEMA (2002), the WTC towers were designed to resist the accidental impact of a Boeing 707 jet airliner. The building design assumed the weight of the incoming object to be 263,000 pounds (119.4 tons), with a speed of 180 mph (80.5 m/sec). Under such a scenario, the kinetic energy at collision could not exceed 3940 ton-m (28470 kips-ft). Unfortunately, the Boeing 767-200ERs that attacked the WTC towers were heavier (274,000 pounds; maximum takeoff weight is 395,000 pounds) and had much higher speeds - 470 and 590 mph (210 and 264 m/sec) than these building design specifications.

In this chapter, the general magnitude of the collision and the factors associated with it are explored. Based on the information available to the public, an estimate of the range of collision parameters was made using classical dynamic impact analysis. In addition to speed and weight, many factors were involved in the collision process, such as the surface of contact, shape of the colliding bodies, the material properties, and duration of collision. Due to the significant uncertainties associated with these factors, the resulting estimate can only aid in understanding the general magnitude of the collision. A more detailed analysis or simulation is needed to obtain an accurate and quantitative measurement.

2.1 Collision Impact Estimation

Collision is a complicated process. The simplest model of collision is the particle-to-particle collision. In this model, collision is characterized by a very significant velocity alteration in a very short time period typically as a result of very large forces. Collision between two impacting bodies can be elastic, inelastic (elastoplastic) or plastic depending on the reactions of the two bodies upon contact. If the reactions instantaneously disappear after contact, the collision is considered elastic. For an elastic collision, the coefficient of restitution is $k = 1$ (absolutely elastic bodies). If the interaction bodies are fully plastic, then $k = 0$. In most cases of inelastic collision, $0 < k < 1$.

The main parameters of the direct central collision can be described by the following three relationships.

1. The common velocity of the bodies at the moment of maximum contact:

$$w = \frac{m_1 v_1 + m_2 v_2}{m_1 + m_2} \quad (2-1)$$

$$\left. \begin{aligned} u_1 &= w + k(v_2 - v_1) \frac{m_2}{m_1 + m_2} \\ u_2 &= w + k(v_1 - v_2) \frac{m_1}{m_1 + m_2} \end{aligned} \right\} \quad (2-2)$$

where m_1 and m_2 are the masses of the interaction bodies, v_1 and v_2 are the velocities before collision, u_1 and u_2 are the velocities after impact, and k is the coefficient of restitution.

2. The total momentum of collision:

$$S = \frac{m_1 m_2 (v_1 - v_2)(1 + k)}{m_1 + m_2} \quad (2-3)$$

3. The loss of kinetic energy:

$$T_0 - T = \frac{1 - k}{1 + k} T^* = \frac{m_1 m_2 (v_1 - v_2)^2 (1 - k^2)}{2(m_1 + m_2)} \quad (2-4)$$

where T^* is kinetic energy of the lost velocities:

$$T^* = \frac{m_1}{2} (v_1 - u_1)^2 + \frac{m_2}{2} (v_2 - u_2)^2 = \frac{m_1 m_2 (v_1 - v_2)^2 (1 - k^2)}{2(m_1 + m_2)}. \quad (2-5)$$

If $k = 0$ (the bodies behave fully plastic), then $T_0 - T = T^*$ (Carno theorem).

If $k = 1$ (the bodies behave fully elastic), then $T_0 - T = 0$.

The momentum and energy applied to the WTC towers by the Boeing 767s can be estimated by using equations (2-3) and (2-4). These relationships, however, can only be used to estimate the shock parameters for the two extreme cases: fully elastic or fully plastic collisions.

In the case of the WTC, the mass of the buildings (about $2.5-3.0 \times 10^4$ ton-sec²/m or $1.68-2.01 \times 10^4$ kips-sec²/ft) was much larger than the mass of the airplane (about 12-18 ton sec²/m or 8-12 kips-sec²/ft). The moving body disintegrated and the towers received local damage. This process is described as "penetrative" impact, the effects depend mainly on parameters of the moving body, and the impact may be considered as fully plastic ($k=0$).

Based on the above, it is possible to estimate the total kinetic energy of the impact as shown in Table 2-1.

Table 2-1. Kinetic energy of the impact

WTC	Airplane	Mass ¹		Velocity (FEMA, 2002)		Full Kinetic Energy		
		kips-sec ² /ft	ton-sec ² /m	mph (kmph)	m/sec (ft/sec)	ton-m x10 ³	joule x10 ⁷	kips- ft x 10 ⁴
WTC1	American Airlines, 11	8.52	12.68	470 (756)	210 (689)	279.5	274.14	202.3
		12.26	18.25			402.4	394.65	291.1
WTC2	United Airlines, 175	8.52	12.68	590 (949)	264 (866)	441.8	433.27	319.5
		12.56	18.25			636.0	623.70	460.0

¹ Full cargo capacity (weight) of the Boeing 767-200E equals 179,170 kg, but according to FEMA (2002) at the moment of attack, the weight of the planes was estimated to be 124,400 kg (274,000 pounds). Therefore, in the table, both data are used.

Table 2-1 shows that the maximum common kinetic energy of the Boeing airplanes at the moment of impact was in the range of 280,000 – 640,000 Tm (2700 – 6200 million joules). All this energy could be transferred into shock energy only in the case of rigid impact, when a moving body (airplane) interacts with an absolutely rigid barrier (building).

The analysis of the penetrations of both towers indicates that collisions were inelastic, since the planes were disintegrated and the exterior building frame systems were damaged. As seen in many photos, the exterior walls were thoroughly penetrated, and portions of debris from the planes were found outside the opposite side of the buildings. In fact, the damage to the exterior walls on the impact side may have played a positive role by further delaying the collapse of the buildings because a portion of the kinetic energy from the planes was dissipated through inelastic deformation of the exterior wall frame.

Since it is impossible to precisely quantify the mass directly involved in the collisions, quantitative estimates in the following section are based on a hypothetical case described below. Note that these assumptions are not based on damage evidence; rather, their purpose is to simplify the calculations. They are:

1. The impact involved 50% of the moving masses (airplanes), (due to fully plastic impact),
2. Half the energy was transferred to shock (the rest was extracted as thermal and other kinds of energy),
3. The disintegration of the airplanes and the building damage began at the moment of contact between the airplanes and the exterior walls,
4. The collision process ended at the instant the airplane (or remaining debris) reached the central core of the towers, or when the debris reached the opposite walls of the towers, and
5. The impulse can be described by a triangular shape loading function, with duration equal to the time between the beginning of contact and the point when the airplane debris reached the opposite walls of the building.

2.2 Momentum and Duration of the Collision Impulse Loading

The impulse loading of the airplane applied to the buildings may be described by equations (2-6) and (2-7):

$$\left. \begin{array}{l} P(t) = P_0 f(t) \text{ when } 0 \leq t \leq \tau \\ P(t) = 0 \text{ when } t > \tau \end{array} \right\} \quad (2-6)$$

where $P(t)$ is the dynamic impact force, and $t = 0$ is the beginning of loading action, P_0 is the maximum value of the impact force, and $f(t)$ is the function of the impulse shape, maximum $f(t) = 1$.

The loading force described by equations (2-6) and (2-7) is characterized by three parameters: duration τ , shape $f(t)$ and maximum force P_0 . P_0 is related to the momentum S , given by

$$S = P_0 \int_0^\tau f(t) dt \quad (2-7)$$

The two major parameters of the impulse are momentum and duration, which are considered in the following discussion.

Momentum of Impulse

In the impact-resistant design of structures, the momentum of the impulse force applied to the structure from the direct shock of an impacting body is determined by:

$$S = m v_0 (1 + k) \quad (2-8)$$

where m is the mass of the body, v_0 is the velocity at the beginning of the impact, and k is the coefficient of restitution.

Then, for plastic impact ($k=0$), the momentum of the impulse is given in Table 2-2.

Table 2-2. Momentum of impulse

WTC	Airplane	Mass ¹ ton-sec ² /m	Velocity (FEMA, 2002)		Momentum	
			mph	m/sec	ton-sec	kips-sec
WTC1	American Airlines, 11	3.17	470	210	665.7	1467.6
WTC2	United Airlines, 175	3.17	590	264	836.9	1845.0

¹ Based on a conservative assumption that only one quarter of the masses were used as "disruption mass." Therefore, only part of the moving masses reached the central cores.

Duration of the Impulses

For the duration of the impulses, the end of the collision is considered to be the point that the disintegration process is completed. Two separate cases are considered:

Case 1. The end of impact is assumed to be the point of the building's central core. For WTC1, the distance is 18.3 m; initial speed is 210 m/sec; $\tau = 0.087$ sec. For WTC2, the distance is 10.7 m; initial speed is 264 m/sec; $\tau = 0.041$ sec.

Case 2. The end of impact is assumed to be the point of the opposite wall of the building. For WTC1, the distance is 63.1 m; speed is 210 m/sec; $\tau = 0.30$ sec. For WTC2, the distance is 63.1 m; speed is 264 m/sec; $\tau = 0.24$ sec.

Maximum Forces of the Impulse

If we consider a triangular shape of impact loading function, then the range of maximum forces applied to the WTC towers are shown in Table 2-3.

Table 2-3. Impact parameters

WTC	Airplane	Momentum (ton-sec)	Duration (sec)	Max. Force (ton)	Max. Force (kips)
WTC1	American Airlines, 11	665.7	0.087	15,303	33,740
			0.300	4,438	9,780
WTC2	United Airlines, 175	836.9	0.041	40,824	90,000
			0.240	6,974	15,375

As seen from Tables 2-1 through 2-3 and based on the assumptions made, the kinetic energy of the planes involved in the building damage was between $70-160 \times 10^3$ ton-m ($5.05 \approx 11.5 \times 10^5$ kips-ft). The momentum was between 665-840 ton-sec ($\approx 1470-1850$ kips-sec), and the maximum forces were between 4,440-40,800 tons (9780-90,000 kips). Using these estimates, corresponding impact responses and the damage process of the buildings are discussed with respect to load-bearing capacity.

2.3 Crash Resistance

The amount of the kinetic energy of the plane absorbed by the exterior steel frame system of the perimeter wall's destruction is examined first. According to FEMA (2002), 36 columns on the building's exterior wall were destroyed over portions of a four-story range. Most of the columns were destroyed at the bolted connections due to shear-bolt fracture (see Section 4.2 on Exterior Wall Frames). The shear capacity of a single bolt is 52.8 kips (24.0 tons). Then, assuming that all columns failed by shear force, the load-bearing capacity of the damaged portion of the wall should be approximately 7,000 tons (15,430 kips). However, for the exterior wall to fracture, the penetrating airplane must overcome frictional forces, including the resistance of the floor diaphragms. Among other factors, the friction forces depend on the magnitude of the normal pressure. For WTC1 and WTC2, the estimated friction forces were 2,000 tons and 3,300 tons, respectively. On this basis, to remove the exterior walls from the impact side of WTC1 and WTC2, the acting forces of the airplanes must have been greater than 9,000 tons (19,840 kips) and 10,300 tons (22,700 kips), respectively. Such forces indicate that the assumed impulse duration (or the assumed magnitude of the "disruption masses" or both) is inaccurate. The end of the impact action was sooner than assumed in case 2, when the Boeing's debris reached the opposite walls of the tower (possibly, only a small portion of the total mass, such as the engine and gears, reached these walls).

The duration of the impulses are estimated based on the assumption described by case 1 (impact ended at the instant that most of the moving mass reached the two towers' central cores). The maximum forces must have been more than 15,000 tons (33,000 kips) for WTC1 and 40,000 tons (88,200 kips) for WTC2.

An estimate of the load-resisting capacity of the interior construction (such as floor diaphragms and frames of the central core) is more complex, and cannot be carried out because of a lack of specific information. If the floor diaphragms are assumed to be rigid, then complete airplane penetration into the building is not possible, and the towers should absorb the collision energy through local damage in the range of one or two modules of the exterior steel frame system. In this case, most of the airplane debris should be outside of the buildings on the impact side. However, photographs of the actual event do not support this conclusion.

If the fracture of the exterior steel frame is mainly caused by the shear failure of most of the bolted connections, it is possible to approximately determine the load-bearing capacity of the exterior wall framing system of the WTC

towers by estimating the energy dissipation of the destruction of bolted-connections and of overcoming frictional forces. Calculations show that the energy-absorbing capacity of all 36 damaged columns does not exceed 1000 ton-m (7230 kips-ft). Given the estimate of the airplane's kinetic energy (Table 2-1), one may conclude that a much greater part of this energy was dissipated inside the buildings after the airplanes had penetrated through the exterior walls.

From the approximate analysis of the possible impact scenarios on the WTC towers given above, the impact of the airplanes could be a significant contributing factor to their eventual collapse because the load resisting structural framework was substantially compromised. Additional loading, such as elevated temperatures or a pressure wave from the fireball, may have further weakened the building and led to the progressive collapse of the towers.

3.0 Fire

Although it is generally believed that fire loading affected the collapse of the WTC towers, there have been discussions on the relative significance of the fire loading on the progressive collapse of the buildings. In this study, no attempt is made to quantify the role of fire in the structural collapse resulting from a combination of loadings. The nature of fire loading on a structure differs from other types of loadings with respect to possible effects on the performance of structural components. Thus, the fire protection design and the fire intensity of the twin towers are reviewed to provide the background for discussion of multi-hazard loading design of buildings and their components in later chapters.

3.1 Fire Protection Design

Fire Protection Systems

Fire protection for the twin towers had three lines of defense (FEMA 2002). The first was the automatic sprinkler protection that was installed in 1990 and was intended to provide quick and automatic extinguishing or confining capacity. The second consisted of manual firefighting capabilities, supported by the building standpipe system, emergency fire department elevators, the smoke control system, and others. The third was fire protection of structural members, which would not be called upon until the automatic and manual suppression systems had both failed. This last line of fire defense, which relates directly to load carrying capacity of the structure, will be introduced in the following section.

Passive Protection

Up to the 39th floor of WTC1, the structural elements were originally protected from fire with a spray-applied product containing asbestos, which was later abated inside the building through either encapsulation or replacement. On other floors of WTC1 and all the floors of WTC2, all the members of the floor trusses were protected with a spray-applied asbestos-free mineral fiber material, which was made of manufactured inorganic fibers, proprietary cement-type binders, and other additives. The average thickness of spray-applied fireproofing on the trusses was $\frac{3}{4}$ inch.

Spandrels and girders were designed to have a three-hour fire resistance with similar spray-applied materials except for the interior face of perimeter columns between spandrels, which were protected with a plaster material.

Upgrade of Fire Passive Protection after the 1994 Explosion

Following the 1994 attack on the WTC, a decision was made to upgrade the fire protection by increasing the fireproofing thickness of the trusses to 1-1/2 inches (applying additional coating material to the trusses). The upgrade was executed in individual floors when they became vacant. By September 11, 2001, a total of 31 stories had been upgraded. All of the floors that were initially damaged by WTC1 had been upgraded, while only the 78th floor in the impact zone of WTC2 had been upgraded (FEMA, 2002).

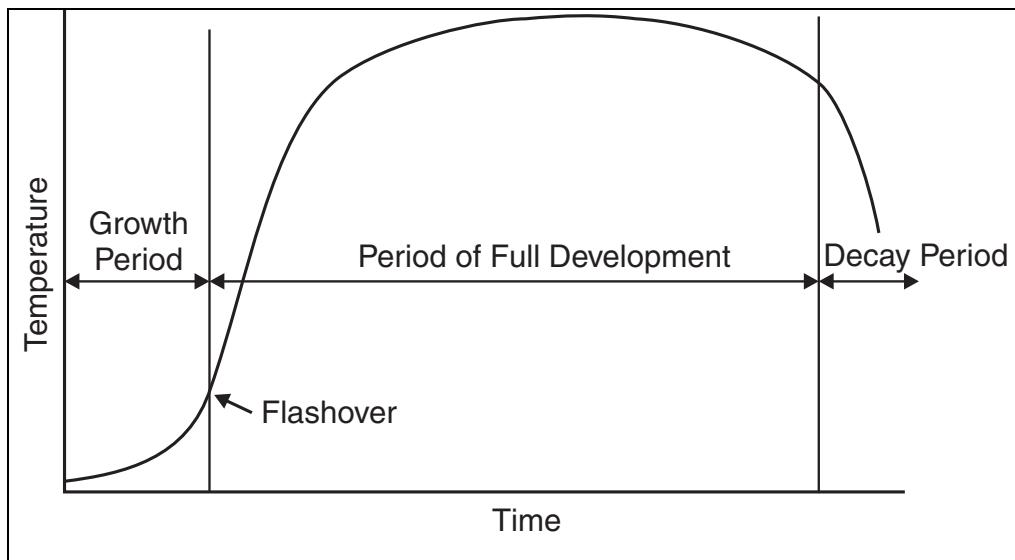
The analysis by Quintiere et al. (2002) shows that the different fire resistances between the two towers may explain why WTC2 collapsed sooner than WTC1. Assuming that truss rods failed first, the times for the estimated failure of the floors in both towers has been computed as 105 ± 20 minutes for WTC1 and 51 ± 9 minutes for WTC2. These time periods matched well with the actual collapse times of 102 and 56 minutes, for WTC1 and WTC2, respectively.

3.2 Estimation of Fire

Fire Development and Severity

Fires in buildings are typically “compartment fires.” The development of a compartment fire can be divided into three periods: growth, full development, and decay, as shown in Figure 3-1 (Lie, 1992; Li et al., 1999). Between the growth period and full development period is a phenomenon known as “flashover.” In this period, the fire makes a qualitative change in its characteristics, which may ignite unburned objects nearly simultaneously.

It is relatively easy to extinguish fire during its growth period, when the burning area is limited and the temperature level and the flow of the smoke are low. In the full development period, the temperature reaches its highest level and the radiation and convection of the heat flux accelerate. The duration of this period is a function of the combustible objects, the size of windows and doors, and the thermal property of the walls. The decay begins when the temperature decreases to a certain level.



Lie, 1992

Fig. 3-1. Development of a compartment fire

Fire severity has been defined as normalized heat load at the room boundaries. The heat load is the total heat absorbed by the room boundaries (per unit surface area) and normalization is achieved by dividing the heat load by the thermal inertia of boundaries. The five factors that affect fire severity (Lie, 1992) are:

- total fire load (total mass of combustibles),
- ventilation parameter (characterizing the rate of inflow of air into the room),
- total area of the room's internal surfaces,
- thermal inertia of the compartment's boundaries (low for insulating materials, high for conductors), and
- fraction of energy of volatile combustible materials released within the room per unit time.

Heat Release Rate

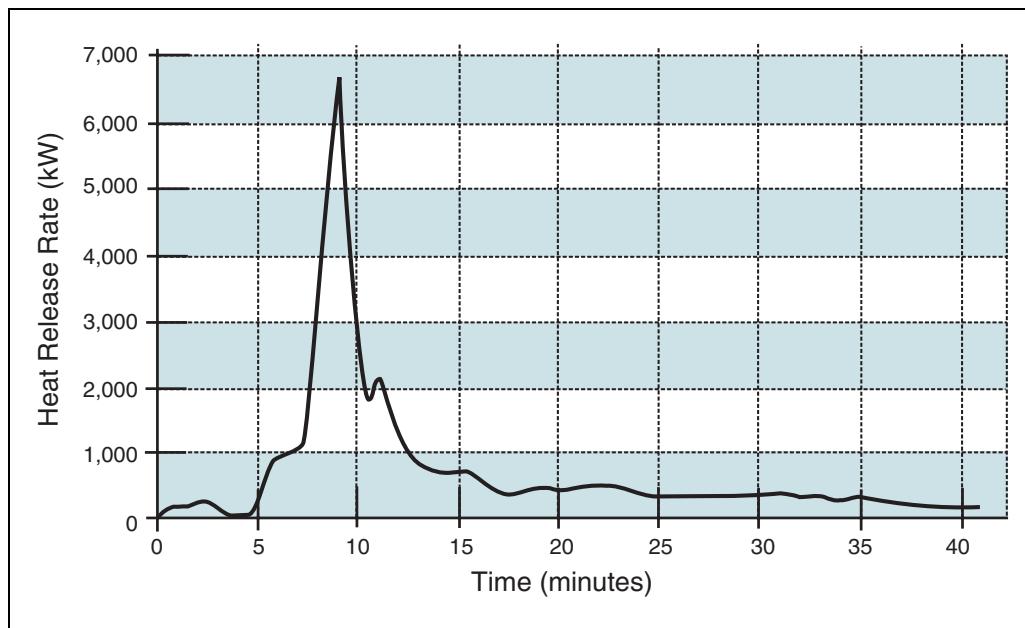
The important parameter that quantitatively describes the intensity of a fire is the heat release rate (HRR), the rate at which the combustion reactions produce heat. The HRR of a burning item is measured in Kilowatts (KW).

HRR, which has been described as the single most important variable in fire hazards, may be calculated by the following equation (Vytenis, 2002):

$$HRR = \Delta h_c \times MLR \quad (3-1)$$

where h_c is the effective heat of combustion ($\text{MJ}\cdot\text{kg}^{-1}$) and MLR is the mass loss rate ($\text{kg}\cdot\text{s}^{-1}$).

HRR can be directly measured from an experiment. As an example, Figure 3-2 shows the HRR for an office module, with the peak value occurring between 8 and 9 minutes.



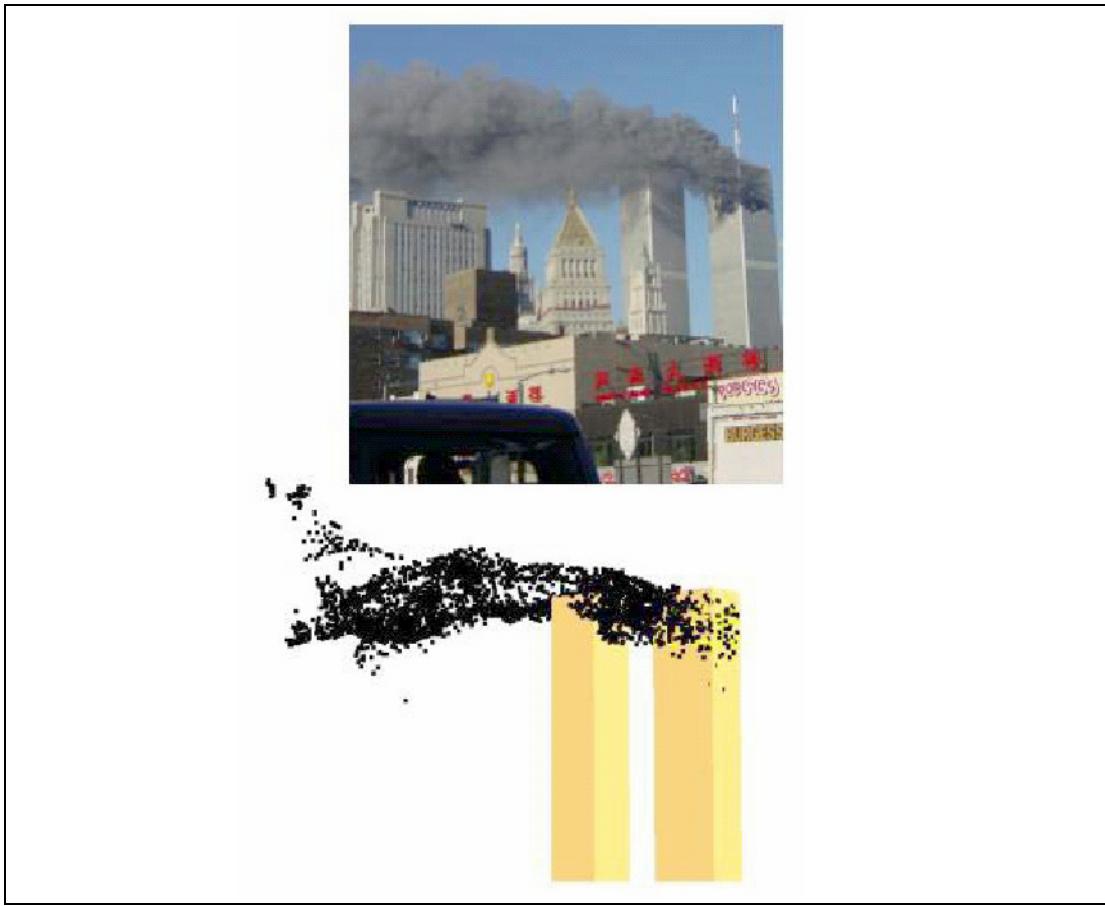
Madrzykowski, 1996

Figure 3-2. Heat Release Rate for office module

Using the Fire Dynamics Simulator (FDS), the National Institute of Standards and Technology (NIST) has simulated the behavior of fire and smoke in the WTC, with a mathematical model (Ronald et al., 2002). The simulated plume trajectory has good agreement with the observed configuration (shown in Figure 3-3), which indicates that the rate of energy supplied to the plume by the fire was on the order of magnitude of a gigawatt (GW). The rate of energy supplied to the plume and the energy-loss rate, including radiation out of each tower, and a portion for heating the structure itself, give the total heat

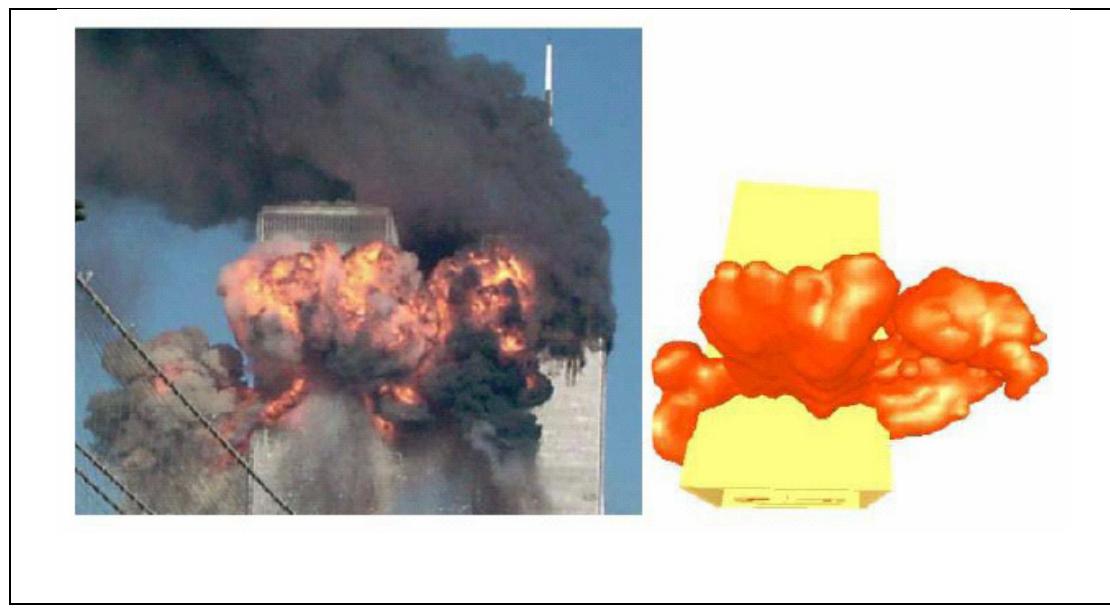
release rate (HRR), which is the most important parameter describing the fire loading in each tower for our purposes.

Assuming that the planes carried approximately 31,000 to 34,000 liters of jet fuel (which is less than 38,000 – 76,000 liters according to FEMA, 2002) and that the plane dumped its entire fuel load over only one floor (splashing all materials on that floor, including combustible and non-combustible matter), Ronald et al. (2002) estimated the fuel burning with a unit heat release rate of approximately $2\text{MW}/\text{m}^2$, yielding a total heat release rate of several GW. At this burn rate, the jet fuel would be consumed in only a few minutes, assuming an adequate air supply, and in less time if the fuel was spread over a larger area.



Ronald et al., 2002

Figure 3-3. Comparing a photograph with the simulated plume for WTC1



Ronald et al., 2002

Figure 3-4. Comparing a photograph with the simulated fireball for WTC2

Figure 3-4 (Ronald et al., 2002) shows a comparison of a photograph of the observed fireball and a corresponding picture taken from a simulation of this fireball a few seconds after the collision for WTC2. The fireball is simulated by introducing a very large heat release rate over a short time so that the total energy released during the life of the fireball conforms to the amount estimated from videos of the event. Technically, the fireballs are found to be deflagration waves of the mixed and burning jet-fuel/air combination immediately after the collision. This conclusion is reached because the flame propagation speed of the fireballs observed from videos approached only a small fraction of the speed of sound in ambient air.

Many studies have attempted to estimate the temperature of the twin towers during this event. Some estimates put the temperature at the melting point of steel, approximately 1,400° C (2,500° F). However, the melting point varies with the steel alloy used. Thomas and Christopher (2001) suggested that the fire temperature in the twin towers could not be as high as 750~800° C (1400~1500° F). For carbon burning in pure oxygen, the maximum is 3,200 ° C (5,800° F), which would be reduced by two thirds if air were used rather than pure oxygen. Thus, the maximum flame temperature increase for burning hydrocarbons (jet fuel) in air is estimated to be no more than 1,000° C (1,800° F) because it is not easy to attain the best ratio of fuel and air to reach the maximum temperature with a diffuse flame. Thomas (2002) suggested that the WTC fire was probably 700° C (1,300° F) or 750° C (1,400° F), because the

fire had melted the aluminum in the airplane's structure and aluminum has a melting point around 650° C (1,200° F). This estimate is supported by the Silverstein report (Post, 2002), which also concludes that the fire temperatures in the twin towers were lower than typical "fully developed" office fires.

A microstructural analysis of a section of an A36 wide flange beam retrieved from the collapsed WTC7 has shown that the temperatures in the region of this steel beam had approached ~1000° C (1,800° F), forming the eutectic liquid by a process similar to making a "blacksmith's weld" in a hand forge (Barnett et al., 2001).

From the above analysis, the fire temperatures in most of the burning spaces of the twin towers should have been about 600~700° C (1,100~1,300° F), and in some small regions the temperature may have been higher than 700° C (1,300° F). For unprotected steel members, their temperatures can rapidly increase to fire temperatures. In the twin towers, the collision, field of flying debris and fireballs probably compromised spray-applied fire protection of some steel members in the vicinity of the collision, so that their temperature may have reached a level close to the fire temperatures.

Ryder et al., 2002 examined the effect of the loss of fire protection material on the fire resistance of steel columns based on a three-dimensional finite element heat transfer analysis. From the simulation results, the area of the missing protection and the size of the column have an appreciable effect upon the fire resistance of the column regardless of the protection thickness. Further, the temperature rise in the column is primarily sensitive to the amount of missing protection, with the size of the column gaining significance only later in the test.

3.3 Structural Damage Related to Fire

When temperature rises in a steel structure, a redistribution of forces will occur as a result of the reduction of the elasticity modulus and strength, and the additional and differential thermal forces induced by the expansion of restrained steel members. Relevant information of the temperature-dependent mechanical and thermal properties of structural steel are summarized in Appendix B.

There are a number of analytical and experimental studies concerning the response of steel structures at elevated temperature. These include studies by Nerves et al., (2002), Wang and Davies (2003), Zhao and Shen (1999), Milke (1999), and Li and Jiang (1999).

Thermal Expansion of Floor Framings and Slabs

Because of the difference in thermal expansion properties, when the concrete slab and its supporting framing are heated, complicated interactions between the framings and slabs will take place. This fact alone can cause a loss of structural integrity between the slab and the supporting frames. Furthermore, the expansion of the floor system as a whole can introduce large thermal forces because of the restraint from the columns. These large thermal forces may cause the shear failure of the connection bolts. The large thermal expansion may also induce outward deformations and apply extra moments to the columns.

Sag Effect of the Floor System

The deflection of steel beams at elevated temperature includes both thermal bowing deflection and mechanical deflection. The thermal bowing deflection is caused by a non-uniform temperature distribution in the beam. The mechanical deflection is a result of an increase in the beam deflection caused by reduced steel strength and stiffness at elevated temperatures. At low temperatures (less than 500° C (930° F)), beam deflection is mainly controlled by thermal bowing. When the temperature is higher than 500° C (930° F), the mechanical deflection may dominate because the stiffness and strength decrease more quickly.

A catenary action can be developed by the floor system because the slab and the supporting framing can lose rigidity at high temperatures. The progress of the catenary action changes the primary collapse mechanism of floor slabs from a flexural to a tension field. This tension field membrane action can increase the vertical load carrying capacity of the floor slab (Rose et al., 1998; Bailey et al., 2000).

In the WTC towers, the floor trusses were connected to the exterior columns with two 15.9 mm (5/8 inch) and two 25.4 mm (1 inch) diameter bolts and with two 15.9 mm (5/8 inch) diameter bolts to the interior columns. This connection is sufficiently strong to support the design load, but it may not be sufficient to support the extra tensile strain resulting from the catenary action.

Buckling Strength of Columns

The collisions of the airplanes with the towers caused massive damage to the perimeter structural systems. For WTC1, photos show that 31 to 36 columns were destroyed over four stories (FEMA, 2002). For WTC2, 27 to 32 columns were destroyed over five stories. The loss or damage of those columns caused load redistribution by increasing the stresses in the remaining columns. Furthermore, failure of connections between the floor framing and the

columns increased the unsupported length of the columns and resulted in a significant reduction of their critical buckling loads.

In the immediate region of collision, the spray-applied fire protection of some columns must have been compromised by the collisions and the subsequent fireballs. As a result of the temperature rise, the critical buckling loads will decrease because of the reduction of the stiffness and the material strength. In addition, the columns may be subjected to significant additional beveling ($P-\Delta$ effect) caused by the thermally expanded steel beams. This effect can be important and it has been demonstrated by the results from a compartment fire test at Cardington (Bailey et al., 1999, 2000; Wang 2000).

The above discussion briefly summarizes what is believed to be the important contributing factors of reduced column buckling strength initiated by elevated temperature that led to the eventual collapse of the twin towers.

Compression Buckling of the Truss Rods from Reduced Modulus of Elasticity and Restrained Elongation

According to the study of Quintiere et al. (2002), the 27.7 mm (1.09 inch) diameter truss rods are the weak links because they have the smallest cross-sectional areas. One suggested scenario of collapse initiation is that the compressed diagonal rods buckled first because of the reduced stiffness combined with restrained elongation at elevated temperature.

The Quintiere report mentioned possible structural damage caused by fire and must be viewed from the perspective that fire damage occurs from an already damaged state caused by the airplane collision. That damaged state is not clearly understood. What is relatively clear, however, is that a sustained high temperature is a likely factor in the delayed collapse of the already damaged steel twin towers. Nonetheless, it remains difficult to conclude that the airplane impact and the high temperature are the only primary factors responsible for the collapse of the twin towers. To develop design approaches for multi-hazard resilient buildings by learning from the WTC disaster, at least one additional loading condition should be examined. That loading is associated with the explosion.

It is noted that the WTC towers were designed against accidental collision of a Boeing 707 jet, and that they were also designed against compartment fire as an independent loading. Explosion after the jet collision was not considered (Robertson, 2002). The effect of the combustion of the jet fuel after the collision of the Boeing 767s will be examined in the next chapter.

4.0 Explosion

This chapter discusses fire-induced vapor cloud explosion (VCE) loadings. Considering the worst scenarios of potential VCE explosion loadings, a few possible modes are estimated for the WTC event and their possible damaging effects on the structural members are explored. Many investigations to date suggest that the combustion of the jet fuel and the fireballs that developed immediately after the impact did not form an explosion. While this is still under investigation, it is helpful to examine the possible worst case scenario that could have happened to the towers.

At the moment the two Boeing 767s crashed into the WTC buildings, in less than 1 second, failure of the fuel tank associated with the flame could have caused a VCE inside the building. As described in Wilson (2001), “we can all no doubt vividly recall that the collisions of the airplane with the buildings caused violent explosions as the fuel tanks, which contained 90,000 liters of aviation fuel on take-off, exploded like enormous Molotov cocktails.” On the other hand, the explosions could have been caused by the strong crash with a rapid diffusion of the flame. Based on the speed of the airplane at the moment of collision, the potential flame propagation rate could be as high as 260 m/s (the speed of the crashing airplane). However, considering the limited time duration for the fuel to mix thoroughly with air, the fireballs witnessed by many observers and recorded by video could be the result of the burning of rapidly spilling fuel, resulting in a flash fire. This opinion is supported by the relatively long (two seconds) duration of the fireballs. Information on what occurred inside the building immediately after the crash would help to explain the nature of the fireballs. In either case, the rapidly expanding fire caused by the sudden forced release of confined fuel may reasonably be considered to be an explosion, although in this case, the explosion could not be clearly or quantitatively defined.

Figures 4-1 and 4-2 show the “fireballs” that were propagated out of WTC2. In front of the fire cloud, a large amount of debris can be seen. The amount and pattern of debris spreading seems to be related to an overpressure wave front, a typical VCE characteristic. Figure 4-3 shows the black-fire smoke billowing out and on top of the WTC buildings.



Associated Press

Figure 4-1. Explosion following the plane's impact into the WTC2



Geschwindner, 2003

Figure 4-2. Explosion following the plane impact into the WTC2



Geschwindner, 2003

Figure 4-3. General view of WTC following the attack.

4.1 Estimation of VCE

In this study, the explosion design theories of Bodansky et al. (1974) are used to estimate the VCE in the WTC towers. Using this approach, two parameters must be given: (1) the total quantity of the jet fuel carried by both airplanes at the time of the collisions and (2) the percentage of the unused fuel involved in the explosion. According to FEMA (2002), at the moment of impact, each airplane had about 10,000 gallons ($\approx 38,000$ liters) of unused fuel. The second quantity is more difficult to estimate. Analysis of accidents in oil refinery plants by Strelchuk and Imaikin (1969) suggests that typically, the full volume of fuel is not transferred into the gas-air mixture of an explosion. For most types of petroleum products, the transfer coefficient equals 0.8. Therefore, it is assumed that not more than 80% of the airplane fuel was transformed into a gas-air mixture.

Some relevant background information about VCE and technical information for quantifying the pressures of explosion proposed by Bodansky et al. (1974) are given in Appendix C.

Estimation of VCE Magnitude (Size and Pressure)

In the following analysis, the considered VCE is assumed to have resulted from the explosion of 1,000 to 8,000 gallons of jet fuel. For structural analysis, the main parameters are the maximum pressure, dimension of the high-pressure zone, and duration of the main phase of the explosion.

For a VCE, the over pressure zone exists only inside the explosion cloud. Therefore, it is important to know the size of the zone with maximum pressure.

Figure 4-4 shows the radius of the VCE cloud sphere (lower curve) and the radius of the sphere of explosion product (top curve) as functions of the quantity of exploding fuel. With the maximum estimated volume of fuel (32,000 liters or approximately 8,000 gallons), the radius of the high-pressure zone should be about 54.0 m (178 feet). Figure 4-5 shows that even for a small VCE (8,000 liters or approximately 2000 gallons), with pressure approximately $14 - 16 \text{ kg/cm}^2$ ($\approx 200 - 230 \text{ psi}$), a 100 ft. cloud sphere can result.

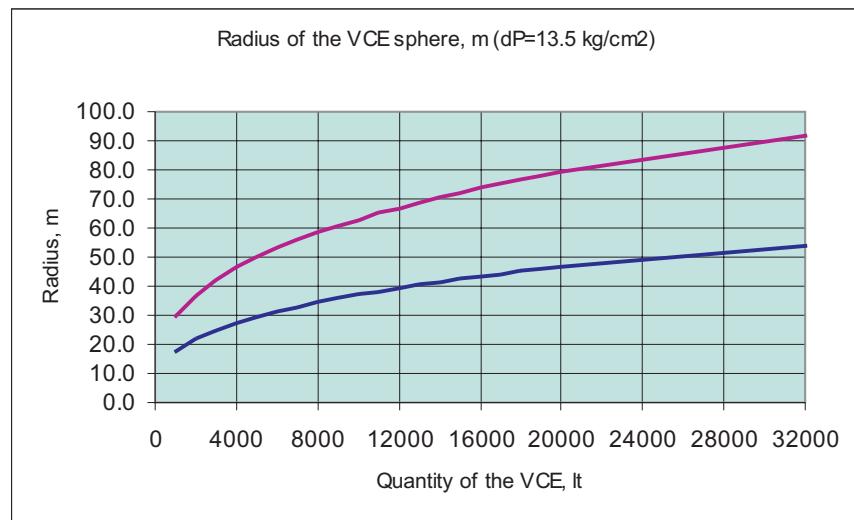


Figure 4-4. Radius of the VCE explosion cloud sphere (lower curve) and zone of explosion product escalation (top curve).

Figure 4-5 shows the dependencies of the pressure distribution versus the distances of the explosion center for different magnitudes of VCE. Note that the radius of the high-pressure zone could be from 27 m (88 ft) for a 1,000-gallon explosion to 54 m (177 ft) for an 8,000-gallon explosion. Outside of this zone, such as the zone of explosion product escalation, the pressure drops quickly. At 100 meters from the explosion epicenter, the pressure does not exceed 20% of the maximum value.

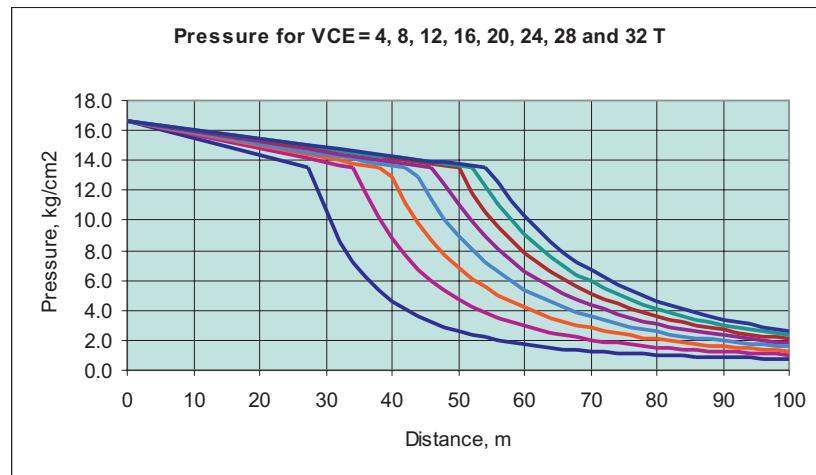


Figure 4-5. Pressure of the VCE explosion for different fuel magnitudes: first curve from left - 4000 liters (\approx 1000 gallons); far right curve - 32000 liters (\approx 8000 gallons)

Figures 4-4 and 4-5 were obtained on the basis of equations C-4 for Zone 1 and C-5 for Zone II, as given in Appendix C.

Estimation of Explosion Distribution

Using the explosion relationship curves in Figure 4-4, the possible range of the magnitude of the explosion for the WTC towers can be estimated.

The cross-sectional dimensions of the WTC buildings were 207 ft. x 207 ft. (63 x 63 m). Explosions occurred inside the building, presumably when airplane debris reached its central core. Therefore, the maximum distance from the center of the explosion to the buildings' exterior wall was from 30 to 180 feet (10 – 60 m).

CASE STUDY: World Trade Center 1

As noted previously, the impact of the Boeing 767 airplane on WTC1 was almost at the center of the north façade of the 94th – 96th floors. Since photo and video recording did not provide any evidence of the explosion-effect or fireballs outside of the building, it is reasonable to assume that the airplane (or plane debris) physically reached the core frame of the building where the jet fuel exploded. Based on this assumption, the epicenter of the explosion in WTC1 could be assumed to be at the center point of the first row column of the building's core. The explosion action inside the building can be depicted in several scenarios, as described in the following paragraphs.

Scenario 1: The explosion was a result of the maximum possible quantity of unused fuel, i.e., 8,000 gallons (\approx 32,000 liters).

Figure 4-6 shows the VCE distribution in the building's floor plan together with two main explosion zones. The center of the explosion is assumed to be the center point of the first row of the core frame. The inner circle is the zone of the explosion cloud; and the outer circle is the zone of the explosion product escalation.

In this figure, the total building floor is within the high pressure zone. The sphere of the explosion cloud is 70% larger than the maximum size of the building floor. The sphere of the explosion product escalation (EPE) is almost three times greater than the building floor dimension. In the case of explosion, the dimension of the fireball should be larger than the one that was observed and reported (FEMA, 2002).

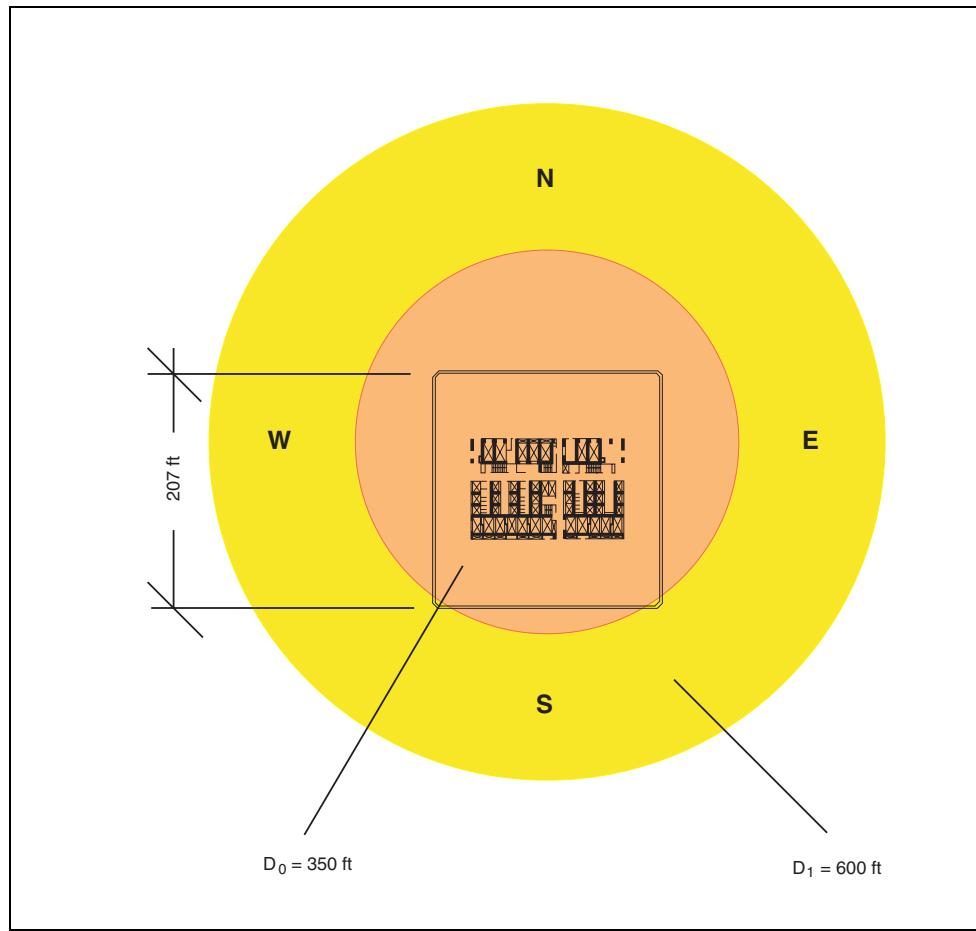


Figure 4-6. Scheme of the VCE = 8000 gallons for WTC1

Scenario 2: The explosion was due to 6,000 gallons of fuel ($\approx 24,000$ liters).

The results of analysis are shown in Figure 4-7. Again, most of the structural elements are within the zone of high pressure. The diameter of the EPE zone equals 540 ft. In this scenario, the dimension of the fireballs outside the building is comparable to the façade size.

Scenario 3: The explosion was due to 4,000 gallons of fuel (≈ 16000 liters).

Figure 4-8 shows the results of the analysis. The structural elements of the north, east and west façades are in the high- pressure zone. The diameter of the EPE zone is 465 ft. and the dimension of the fireballs outside the building is less than the facade size.

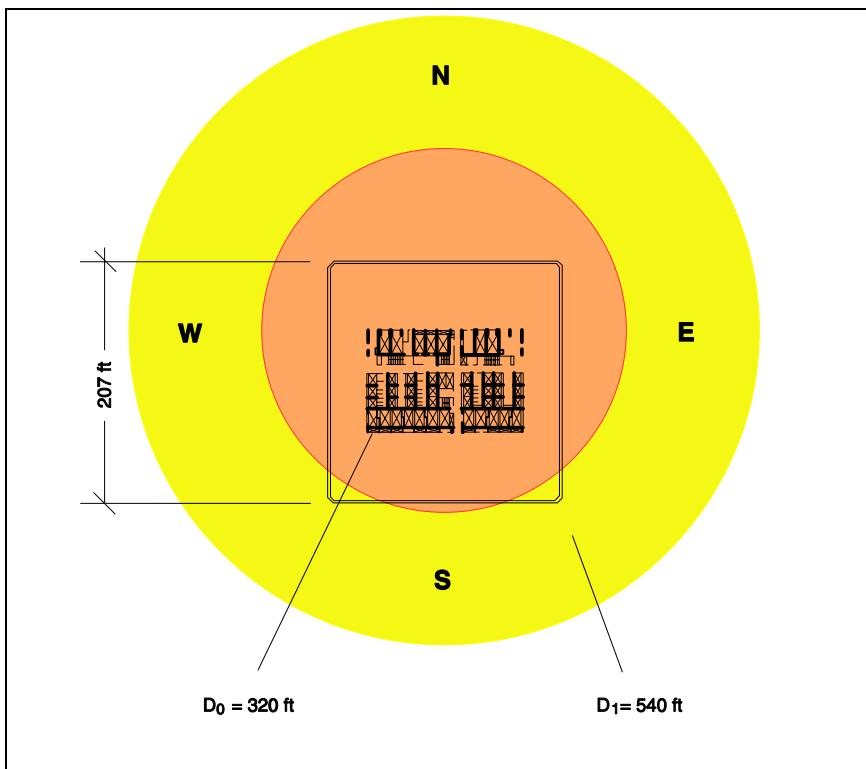


Figure 4-7. Scheme of the VCE = 6000 gallons for WTC1.

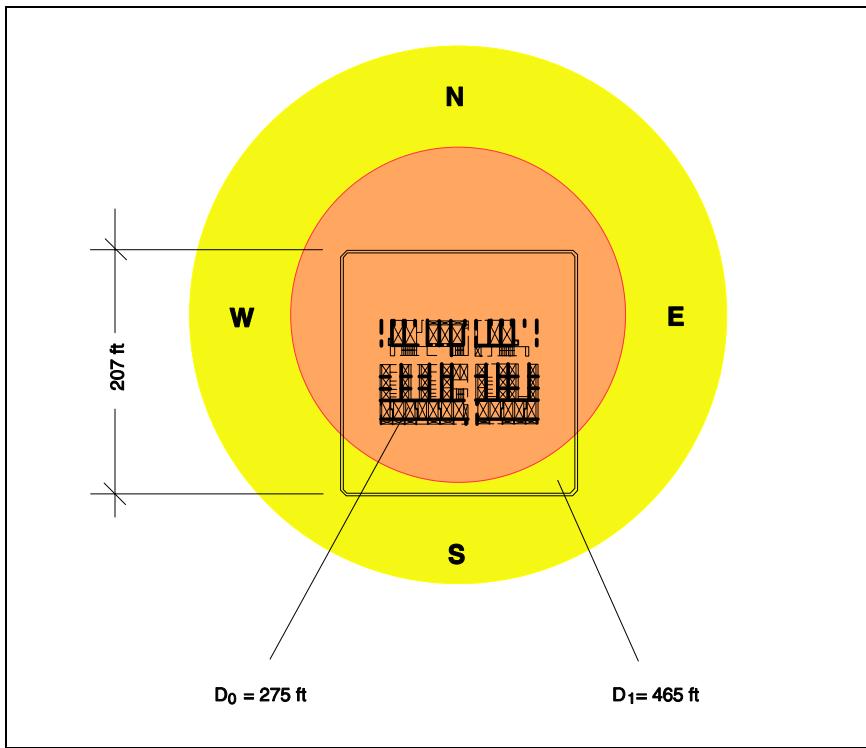


Figure 4-8. Scheme of the VCE = 4000 gallons for WTC1.

Scenario 4: The explosion was due to 2,000 gallons of the fuel ($\approx 8,000$ liters).

Figure 4-9 shows the results of the analysis. Structural elements positioned inside the building are only located in the high-pressure zone. The diameter of the EPE zone is small (340 ft), and fireballs observed and reported by witnesses were larger than those calculated.

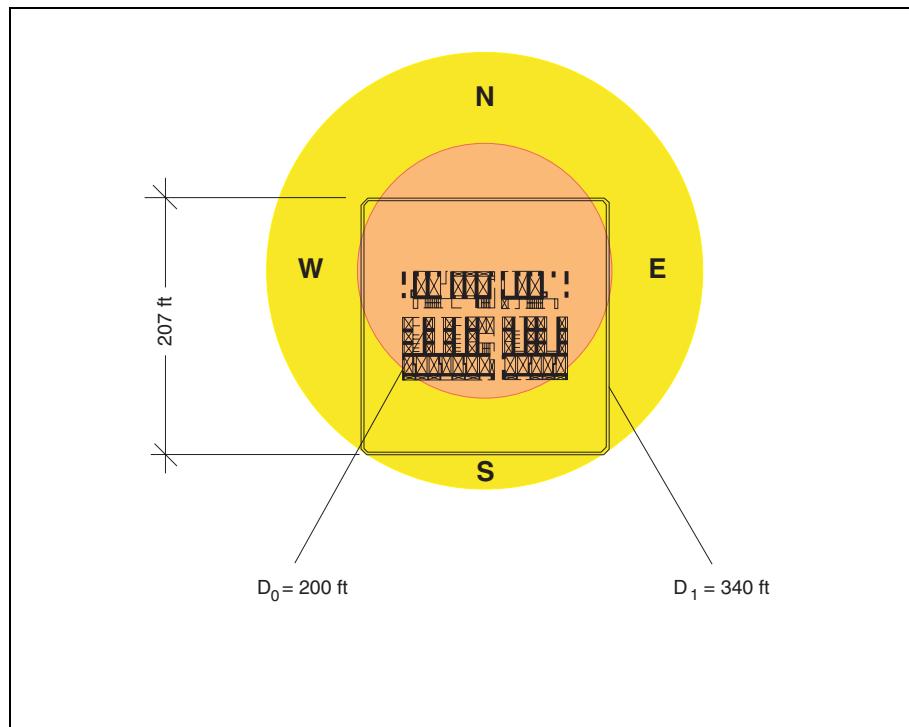


Figure 4-9. Scheme of the VCE = 2000 gallons for WTC1

The preceding analysis of the VCE distribution assumed a point-explosion source. That is, the epicenter of the explosion was assumed to be at the middle of the first row of the building core frame. In reality, the source(s) of the explosion(s) was not symmetric with respect to either the E-W or the N-S directions. For this reason, the size of the fireballs and pressure distributions were asymmetrical on different sides of the building. Nevertheless, the analyses show that for a building with a 207 ft. \times 207 ft. cross-section under VCE of 4,000 – 8,000 gallons of the fuel, most members in the structural load bearing systems are located in the high-pressure zone. This suggests that if the VCE detonation process did happen in the WTC attack, then high explosion pressure could be an important factor for the initiation of the progressive collapse of WTC1.

CASE STUDY: World Trade Center 2

As described in Chapter 2, the impact location of the airplane on WTC2 was approximately at the one-third point of the north facade at the 78th – 80th floors. As for WTC1, it was assumed that airplane debris reached the core frame of the building where the explosion occurred. Thus, it is assumed that the epicenter of the explosion for WTC2 was at the extreme column of the core frame. For an explosion taking place inside the building, two possible scenarios are considered and described in the following paragraphs.

Scenario 1: The explosion was due to 6,000 gallons of fuel ($\approx 24,000$ liters).

Figure 4-10 shows the VCE distribution on the building cross section (based on data from Figures 4-4 and 4-5), together with the two main explosion zones. The center of the explosion is assumed to be the extreme column of the core frame. The inner circle is the zone of the explosion cloud; the outer circle is the zone of the explosion product escalation.

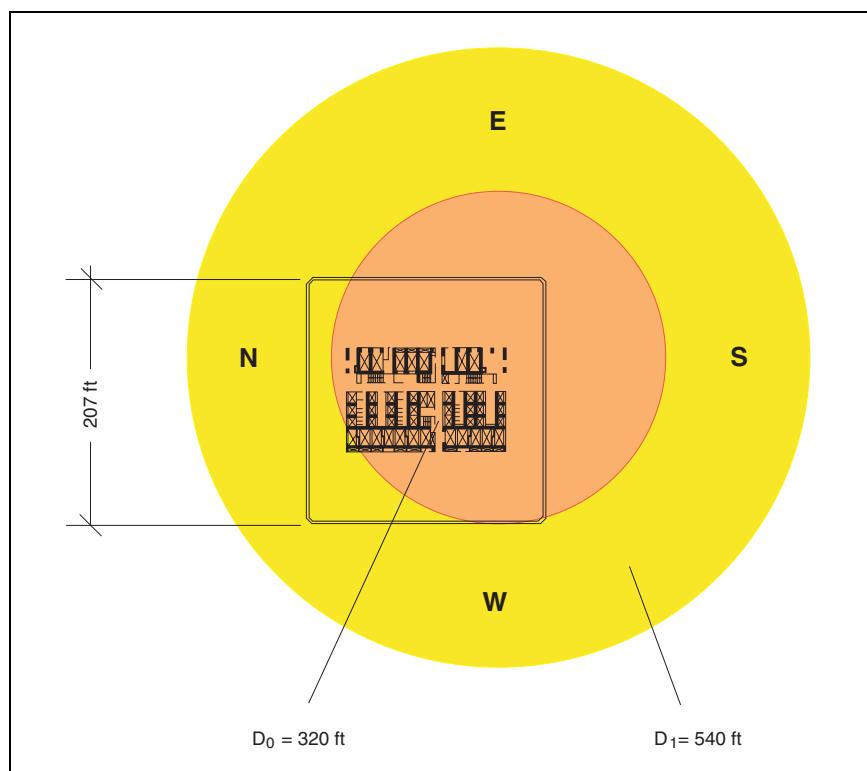


Figure 4-10. Scheme of the VCE = 6000 gallons for WTC2

As shown in this figure, the south, east, and part of the west perimeter wall-bearing systems of the building are within the high-pressure zone. The sphere of the explosion cloud should spread mostly in the south and the east directions. The size of the EPE sphere on the south and east sides compares well with the building size. For this assumed explosion, the dimension of the fireball should be close to that observed and reported.

Scenario 2: The explosion was due to 4,000 gallons of fuel ($\approx 16,000$ liters).

Figure 4-11 shows the results of the analysis. Here, only the east and south sides of the perimeter walls are located within the high-pressure zone. However, the entire building is within the EPE zone.

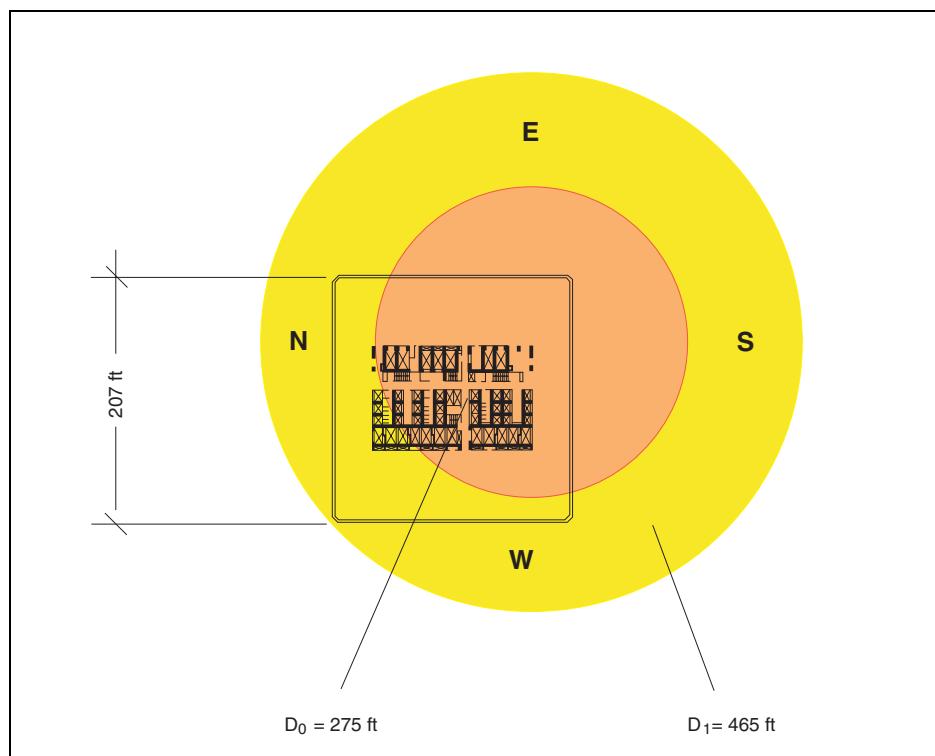


Figure 4-11. Scheme of the VCE = 4000 gallons for WTC 2.

The preceding analyses of the VCE distribution were again based on the assumption that the point-explosion source was at the corner of the core. Since the epicenter of the explosion could be any place in the building, the explosion force's distribution could be different. What can be said here is that for WTC2 with the VCE caused by 4,000 ~ 6,000 gallons of fuel, most of the members of the structural load bearing systems, especially the core frames, and south and east exterior frames were inside the high-pressure zone.

Impact Forces of VCE

The maximum pressure and the pressure distribution are the critical characteristics of the VCE. A full analysis of the structural behavior caused by the explosion requires additional parameters, such as the explosion duration. For practical purposes, triangular “pressure – time” relationships for different explosive materials have been used. Because explosion action is dynamic in nature, the most significant parameter of the explosion is the specific impulse of the explosion. This parameter, which depends on explosion magnitude and duration, is normally used for the dynamic analysis of structures.

Figure 4-12 shows specific impulses of the VCE for different magnitudes of 5,000 – 40,000 liters (\approx 1,300 – 10,000 gallons) of the fuel, with a distance of up to 150 m (\approx 500 ft). As indicated, the magnitude of the specific impulses in the zone of the explosion cloud strongly depends on explosion energy (magnitude). At a distance of 25 m (\approx 80 ft), the effect of the 40,000-liter explosion is twice that of the 5,000-liter explosion. However, at a distance of 50 m (165 ft), the difference is six to seven times greater because outside the high-pressure zone, the magnitude of the impulse decays very quickly.

The analyses show that for a building with a 207 ft. x 207 ft. cross-section under VCE of 4,000 – 8,000 gallons of the fuel, most members in the structural load bearing systems are located in the high-pressure zone. This suggests that if the VCE detonation process did happen in the WTC attack, then the high explosion pressure could be an important factor of the collapse of WTC.

Specific impulses such as those shown in Figure 4-12 provide a measure of explosion energy that can be used for dynamic analysis of structures subjected to explosion loading.

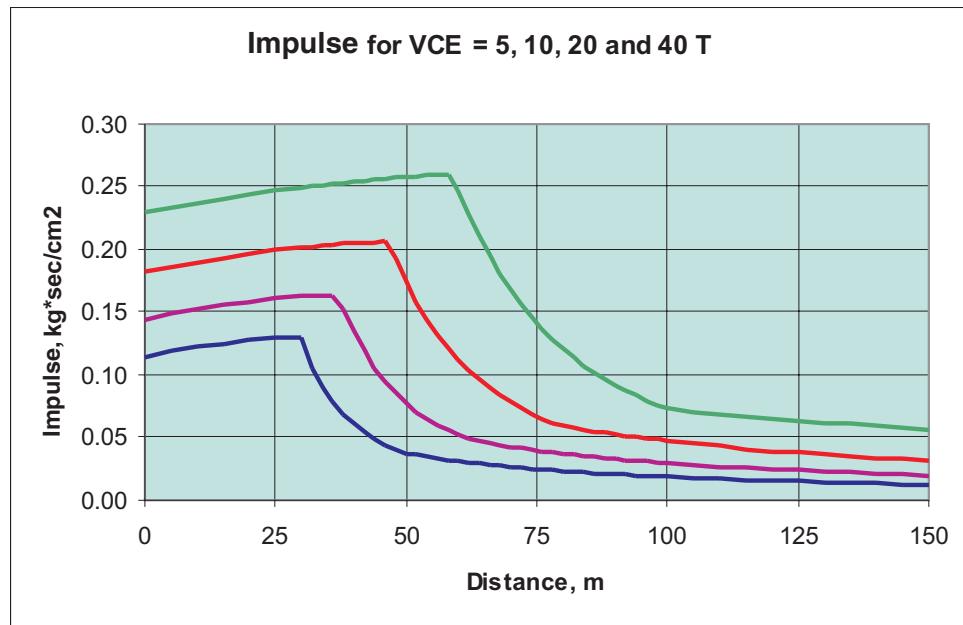


Figure 4-12. Magnitude of the VCE impulse depending from quantity of the explosion's fuel and distance: Top curve –40,000 liters; Bottom curve –5,000 liters

4.2 Possible VCE-Induced Damage

The shock waves of an explosion can produce dynamic loadings of extremely short duration to the structure, and these loadings will not instantly result in a noticeably large deformation of the structural system. However, structural members subjected to the instant high overpressure can be seriously damaged. Structural responses and the mechanics of structural damage under the explosion loading require considerable future research.

In this subsection, the dynamic behavior of a few structural components of the WTC towers under blast loading will be briefly discussed.

Exterior Wall Frames

As described in Appendix A (Figure A-3), the external frames of the upper parts of both towers were formed by box columns with 14 in. x 14 in. sections and spandrel beams (54 in. x 3/8 in.). The perimeter frames were constructed from modules (36 ft. x 10 ft.). High-strength bolted connections were used between modules at the middle of floors in the vertical direction and at mid-span in the horizontal direction. The nominal bolt diameter was 7/8 in. The bolts were A325 or A490, high-strength bolts (FEMA, 2002). Four-bolt connections were used. The external frames of the WTC buildings were designed to resist part of the vertical and most of the horizontal (wind) loads.

For this reason, column connections were placed in the middle of floor height in the "zone of zero moment." Under such a design, the load bearing capacity of the external wall system and frame connections are adequate. Such a design could not be called upon to resist extreme horizontal local forces.

The possible behavior of a single wall frame module (see Figure A-3) under VCE in the high-pressure zone is briefly discussed in the following.

Assume that the windows had been broken by explosion, and that pressure is uniformly distributed on the surface of the columns and the spandrel beams. With a pressure of 192 psi, the load bearing capacity of the bolted column connections is examined for two explosion distributions:

Case 1. Assume the explosion occurred at the floor level and that the pressure will expand only within one story and act directly on the exterior wall columns. In this case, the bending moment is approximately 949.0 kips-in at the connections.

The load-bearing capacity of the four-bolt column connections (FEMA, 2002, Appendix B, Figure B-7) has a moment capacity equal to 1468 kips-in., or 770 kips with respect to the zero-moment point.

Case 2. The same as Case 1 except that pressure is distributed on all structural modules, including columns and spandrel beams.

The surface area of the modules (without windows) is 30,300 in². Common pressure on the modules is 5,820 kips, and linear pressure on columns is 4.49 kips/in.

The bending moment at the connection is $(4.49 \times 144^2) / 24 = 3880.0$ kips-in, which is significantly more than the capacity provided by the four-bolted connections.



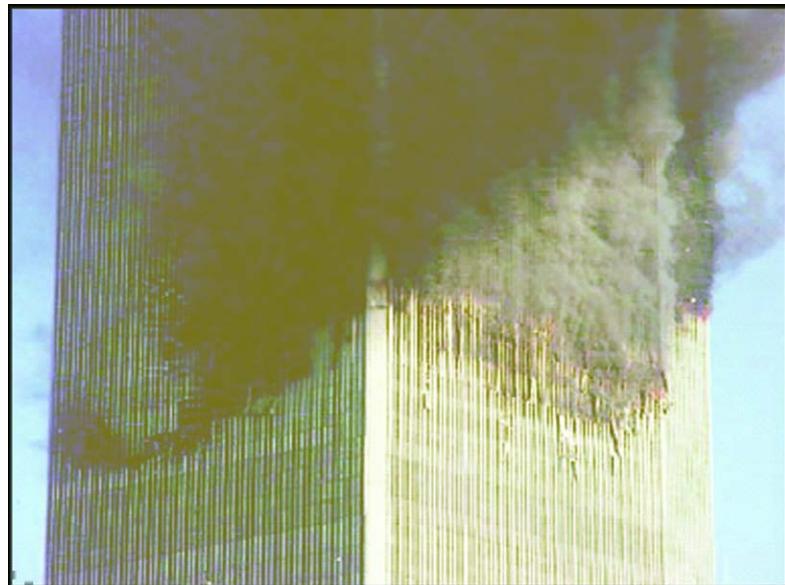
FEMA, 2002

Figure 4-13. View of the fractured columns



FEMA, 2002

Figure 4-14. Fractured bolts at an exterior column



FEMA, 2002

Figure 4-15. Damage to the east side of WTC2

Figures 4-13 and 4-14 show some examples of damaged columns. As seen in both photos, the fractures occurred because of bolt damage. Figure 4-15 shows the damaged exterior wall frame of WTC2. Also in this figure, distortions of many exterior columns can be observed. Although this damage could be attributed to the collision of the airplane, the size of the damage area

in the relatively perpendicular direction of impact leads to the possibility that VCE action could have contributed to the damage.

Floor-Supporting Trusses

As described in Appendix A, the main load bearing floor system of both WTC towers consisted of 60- and 35-foot trusses in the long and short directions, respectively. The trusses were not only used to carry floor loads, but also to transfer the wind load between the exterior wall frame and the core frame systems.

The floor system of the WTC towers was connected to the exterior frames and interior core. Assuming that the floor truss system was located within the zone of a VCE cloud of high pressure, with a uniform distribution of the explosion pressure, an 80-inch uniform line pressure of magnitude of $192 \times 80 = 15.36$ kips/in would be greater than 10 times the design load. Under this scenario, local failure of the floor system (most likely the upper floor, if not both) is inevitable.

In this chapter, the vapor cloud explosion, which could have resulted from combustion of the jet fuel, was examined as a possible but unlikely worst-case scenario of the collapse of the WTC towers. The objective was to provide background information that should be considered in future design approaches for multi-hazard resilient buildings.

5.0 Dynamic Responses of the WTC Towers under Severe Multi-hazard Loadings

5.1 Summary of the Severity of the Multi-Hazard Loadings

Many studies and publications have addressed the collapse of the WTC towers; and the lessons learned from this event cover a wide range of issues. Within the scope of this report, airplane collision, fire, and possible explosions are considered as the direct factors contributing to the collapse of the towers. The magnitude and severity of each of the hazards (collision, fire, and explosion) associated with the WTC disaster are summarized below.

Collision:

- The WTC towers possibly received energy from the collision impact of the airplanes of 2700 million joules.
- The maximum force of the collision was approximately 4,500 ~ 7,000 tons (for an estimated duration of 0.3 seconds).
- Many columns were demolished or seriously damaged by the collision impact.
- Approximately 6,000,000 kips-ft bending moment was applied at the base of the building due to the collision impact at around the 70th floor.

Fire:

- The fire from the aviation fuel peaked at 3 ~ 5 trillion BTU per hour (1 ~ 1.5 gigawatts).
- Temperature rose to a maximum of 700° C (1,300 ° F) in some areas of the structures.
- 400×10^9 joule of total energy was released from the fire of 10,000 gallons of fuel.

Explosion:

- About 10,000 gallons of fuel might have been distributed into a flammable vapor cloud, which caused the large fireball and possibly generated a vapor cloud explosion (VCE).
- About 2000 ~ 6000 gallons of fuel was consumed in the fireball.
- The VCE that resulted from the 2000 ~ 6000 gallons of fuel could have had a radius of cloud spheres ranging from 100 ~ 160 feet. The radius of escalation spheres for the towers could have ranged from 170 ~ 270 feet.

5.2 Structural Response and Behavior of Critical Structural Components Under Severe Hazard Loading

In general, it is understood that the combination of collision impact, subsequent fire and explosions caused the WTC towers to collapse. However, the immediate post-impact time history and accumulative structural responses to each of the hazard loadings are too complicated to be understood at present. They are the subject of a continued investigation.

At the initial crash of the airplanes, both towers survived the impact and did not collapse. Various pieces of evidence, such as large crashed area, knocked off columns and falling debris of building contents, show that the strong impact forces had weakened the load-carrying capacity of the major structural components. Further, the fireball and the associated overpressure wave damaged more columns and beams, and the fire protection wrapping. The high temperature of the subsequent fire produced large thermal stresses and deformations and eventually weakened the strength of the load-carrying members of the structure.

The following discussion provides a brief description of the behavior of some critical components, which were singled out by many studies as potentially triggering the eventual collapse of the towers.

Failure of Connection Between Steel Trusses and Perimeter Columns

The connection failure between steel floor trusses and perimeter columns is suggested to be an initiating point of collapse (Eagar and Musso, 2001; Clifton, 2001). Figure A-9 in Appendix A shows the detailed connection.

In Eagar's opinion (2001), the connection between the trusses and perimeter columns had two, three or four times the strength they originally needed, which was well within the design value. Under the unusual loading

conditions of September 11, 2001, the connection seems to have become the weak link.

From the airplane crash, the core region of all the floors above the contact zone would have sagged downwards due to the partial loss of vertical support in the region. The sagging would have progressed with subsequent thermal expansion of the floor systems and the reduction of the strength and stiffness of the core steel members supporting the floors above from rising temperatures. The progressive sagging could have caused the failure of the connections between the floor trusses and the columns, one of the weakest links (Clifton, 2001).

Column Buckling

Post (2002) offered the opinion that the collapses of the WTC towers were due to the initial failure of the columns rather than the sagging accumulation of the floor trusses. Figures 5-1 and 5-2 show damaged columns that were stripped of fireproofing by the impact of airplanes on WTC1 and WTC2. The report presents results from computer models attributing the failure of columns that either lost fireproofing or were destroyed by the impact of the plane as the reason for the building collapse. The report also shows evidence of the smoke that emanated from the burning towers before the collapses. If the floor trusses had collapsed first, there would have been a mass of smoke, as compared to differentiated smoke, floor by floor.

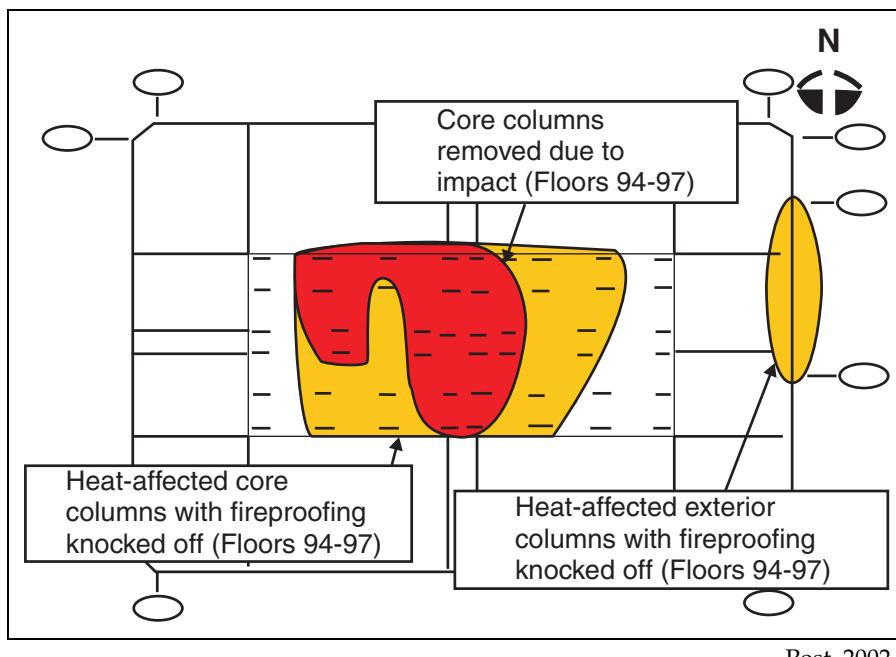


Figure 5-1. Columns' damage from impact in WTC

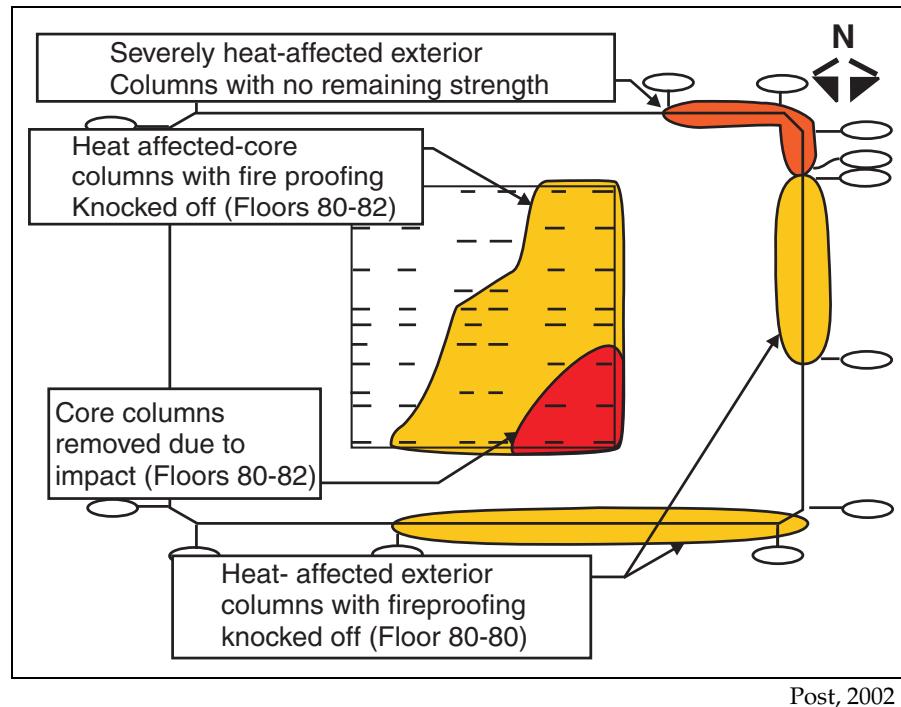


Figure 5-2. Columns' damage from impact in WTC2

Failure of the Truss Rod

Quintiere et al. (2002) suggested that the failure of the truss rods, rather than connection failure, or column buckling, was the initiating event because these rods have the lowest steel cross-sectional mass and thus attained the high level of temperature the fastest. Following this thought and adopting the CIB correlation for the fire (Quintiere et al., 2002), an analysis was made regarding the temperature of the truss rod using an estimated fire temperature of 900° C. Based on a failure model of first-truss compression rod-buckling, the computed times for the estimated failure or incipient collapse of the floors in both towers has been established at 105 ± 20 minutes for WTC1 and 51 ± 9 minutes for WTC2. As noted in Chapter 3, these correlate well with the collapse times of the towers of 102 and 56 minutes, respectively. Furthermore, the difference between the collapse times of the two towers also strongly correlates with the different insulation thicknesses of the steel truss members.

5.3 Evolving Process of Collapse

The process by which the buildings were damaged may be explained in three stages: airplane collision, explosion, and fire.

Damage from Airplane Crash and Jet Fuel Explosion

The collisions of the jets with both towers was followed by building penetration, which allowed part of the kinetic energy of the impact to be absorbed through a local inelastic damage process. On the other hand, at least two floor diaphragms and the exterior walls were disintegrated and the debris represented a significant overload of the floor system. This overloading is also a possible or contributing factor for the building collapse. Additionally, there are other overloading possibilities coupled with the strength reduction (temperature effect) of the structural members.

Many columns in the exterior wall frame system were demolished (31 to 36 on the north facade of WTC1 and more than 25 on the south facade of WTC2) and the remaining columns in the collision zone, and columns below the collision zone, were significantly overloaded more or less uniformly. For WTC2, the impact was asymmetrical and the impacted zone was about 15 floors down when compared with the impact of WTC1. Therefore, the remaining exterior columns of WTC2 after impact and explosion were subjected to a more severe and uneven overload. One might suspect that the southeast corner of WTC2 reached its critical condition first, before the effect of fire temperature became significant.

Under normal conditions, the core structural system carried about 60% of the towers' vertical load. Assuming that a few core columns were destroyed (in WTC1 in the center and in WTC2 in the southeast corner), the after-collision stability of the towers depended mainly on the load-bearing capacity of the remaining core frame system, which was already overstressed. It should be noted that the diagonal brace systems at the top of the towers played a positive role by redistributing certain vertical load from the damaged exterior structures to the core frame system and remaining exterior columns.

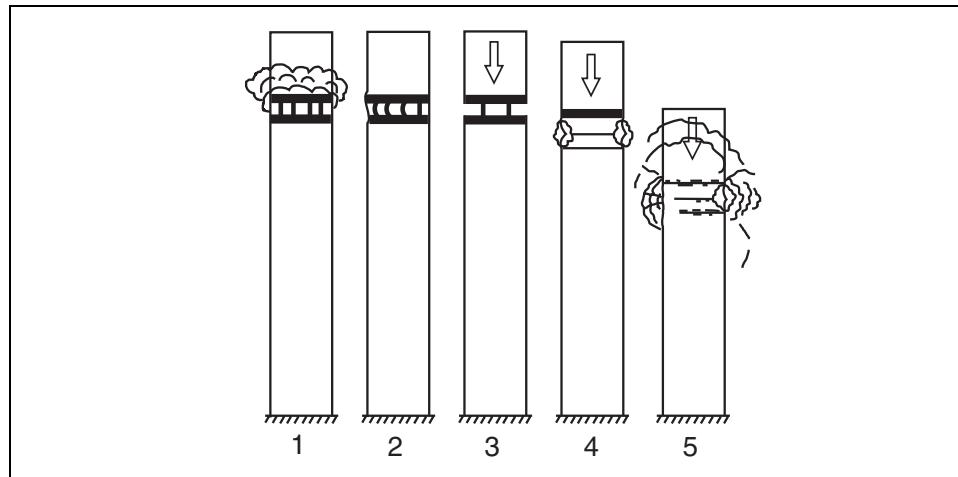
Damage Accumulation

Immediately after impact, the processes of stress relaxation and redistribution (deformation development under critical load) in the structural elements (exterior columns, floor trusses, core columns) began. Additionally, high temperature led to the decrease of the load-bearing capacity of certain structural elements, and the differential temperature distribution added further overstress to certain regions of the already damaged structure. These processes continuously reconfigured the overall structural characteristics of

the buildings until collapse. This weakening process took 102 minutes for WTC1 and 56 minutes for WTC2. For WTC1, the concentration of deformation was more or less symmetrical relative to the building's center of mass and occurred primarily in the core area. However, because the damage zone for WTC2 was in the southeast corner, the process of deformation under the load was different; the tower gradually leaned to the southeast side; hence, the center of gravity gradually moved in the tilt direction.

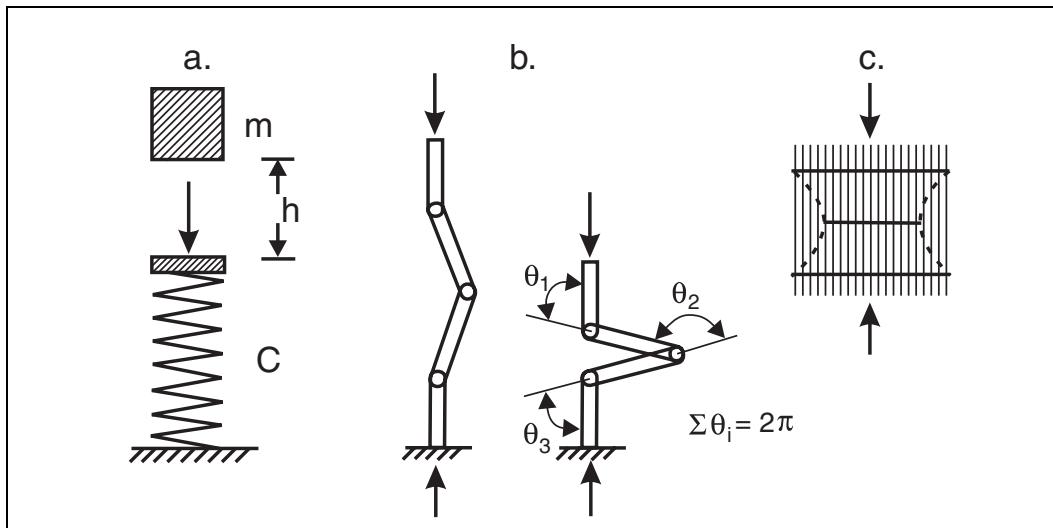
Progressive Collapse Mechanism

Despite the various possible initiating mechanisms of collapse, once the progressive collapse started, both buildings "pancaked" into debris within 10 seconds. The collapse mechanism (domino effect) has been illustrated by Bazant and Zhou (2002). Figure 5-3 shows the stages of collapse of the building. Figure 5-4 is a model for enabling the stages, attributing the collapse to the supposition that parts above the impacted floor fell onto the floor below, with an impact force significantly greater than the floor could withstand. The impact force is estimated to be about 30 times greater than the design load capacity of the floor. The time required for free-fall without any constraints would be approximately eight seconds (versus the observed 10 seconds with undetermined amount of constraints).



Bazant and Zhou, 2002

Figure 5-3. Stages of collapse of the building (floor height exaggerated)



Bazant and Zhou, 2002

Figure 5-4. (a) Model illustrating the impact of the upper part of the building on the lower part; (b) Plastic buckling mechanism on one column line; (c) Combination of plastic hinges creating a buckle in the tube wall

In support of the pancake type of progressive failure without collapsing sidewise, Thomas (2002) noted that the base of the tower was 208 feet to a side, so the tower had to be pushed laterally at least 100 feet for the tower to tip over. That would require a significant amount of bending, for which the columns of the building would have buckled long before developing the large bending movement to tip the tower.

5.4 Lessons Learned

Thus far, the possible causes of the collapse of the WTC towers after the airplane collisions have been reviewed and examined from the viewpoint of structural engineering. The discussion has not been exhaustive and typically excludes many issues related to non-structural components, such as the redundancy of the egress system and the sprinkler system; firefighting and evacuation technologies and practices for high-rise buildings; control of fire spread in buildings with potentially large open floor plans; occupant behavior; evacuation technologies and practices for tall buildings; command, control and communication systems for emergency response; and other important topics.

What are the lessons learned for structural engineers? Although not particularly enlightening to start, the first lessons learned are the “obvious” lessons which structural engineers were already familiar with (see Chapter 6 for an expanded discussion of lessons learned), for example:

- *Redundancy and robustness of the steel framing system*

WTC1 and WTC2 remained standing for a sufficient time after the planes crashed into them, which undoubtedly saved many lives. The resistance to the crash impact greatly benefited from the redundancy and robustness built into the steel framing system. In addition, adequate egress stairways that were well marked and lighted may have contributed to the successful evacuation of thousands of people.

- *Redundancy of connections between trusses and perimeter columns and their behavior in fire*

The redundancy of the connections between steel trusses and perimeter columns has been questioned. Though this redundancy is sufficient to resist the normal design loading, the redundancy of the connection, especially its adequacy in elevated temperatures, should be investigated in detail.

- *Fire proofing*

The fire proofing, including its thickness and ability to resist the impact and explosion, has been the most controversial issue. Fire protection may have played a significant role in the time duration before the collapse of the WTC towers, including the fact that WTC2 collapsed sooner than WTC1.

- *Type of steel floor truss system and its structural robustness and redundancy*

The effectiveness of the steel floor truss system has been questioned (NOVA, 2002).

- *Pattern of Connecting Lines of the Exterior Frame*

An interesting issue at www.Scieneering.com/index.html was raised about how the three-column panels of the exterior wall were welded. These panels were configured in a down-down-down-down or up-up-up-up staircase pattern (shown in Fig. 5-5), which was not effective for failure along the diagonal direction.



Bruneau et al., 2002

Figure 5-5. Three-column panels in down-down-down-down pattern

These ideas briefly illustrate several lessons one may learn from the WTC tower collapse under severe multi-hazard loading conditions. They serve as a reminder to structural engineers about the necessity of fire proofing for steel buildings, importance of structural integrity, redundancy and alternate load paths, as well as the importance of "strong" connections.

These are important but relatively "local" issues in structural engineering. With the magnitude of the WTC tragedy and the severe multi-hazard loading that took thousand of lives and destroyed the two signature buildings, there should be more significant questions asked and more important lessons learned. Certainly, the basic structural engineering design principles should be revisited from a much broader perspective of major disasters beyond constraints of resources, functionality, or even "no collapse."

The authors, as a group of earthquake engineers, have been pondering this issue since the beginning of this study. As a result, they have proposed a new approach for the development of multi-hazard resilient buildings from the lessons of the WTC tragedy by extending the performance-based engineering approach. This approach is outlined in the following chapter.

6.0 Learning from Earthquake Engineering: Some Concepts for Multi-hazard Engineering of Buildings

From a structural engineer's point of view, the WTC towers collapsed mainly because of extreme loadings: a combination of plane crash, fire, and possible explosion. Exacerbating factors included limitations of building fire protection materials, fire suppression systems, and load carrying capabilities after impact. As described in previous chapters, each of the hazardous conditions (impact, fire, fireball or explosion) can be a significant threat to the building structure. However, the simultaneous combination of these hazards created a catastrophe of a magnitude that was truly beyond the thoughts of planners and designers. While it is impractical to design a building, even a landmark building, that retains structural integrity against such extreme loadings, it is nonetheless possible to design structural systems that protect lives by slowing collapse in cases where building collapse cannot be averted. This chapter draws on a performance-based earthquake engineering design methodology to suggest future directions for multi-hazard engineering research and design of buildings to protect lives in circumstances where the emergency evacuation of building occupants is the highest priority.

Despite the human tragedy of September 11, an engineering study of the WTC collapse provides valuable information on future methods for protecting buildings against collapse. This analysis is especially important because current building codes and recommended design provisions (for nonmilitary buildings) do not offer guidance on expected building performance for events of this severity - for any of the hazards understood singly (impact, fire, explosion) or in combination. To meet the public expectation for safety from future terrorist attacks as well as catastrophic natural disasters, it is important to find cost-effective measures for saving lives in such events, even if impact, fire, explosion, and eventual collapse cannot be avoided. Had those systems existed in the WTC towers, many more lives of the building occupants and rescue workers may have been saved on September 11.

To protect future structures against disasters such as the WTC event, planners and structural engineers should pursue a number of new methodologies, engineering models, research directions and technical solutions that go beyond the boundaries of current knowledge, guidelines and engineering practices. After a catastrophic event, the design and performance criteria for

buildings can change. For example, the September 11 event suggests that the “delayed collapse” of a building may be an important design consideration for other heavily occupied buildings. This concept is one of the more important lessons learned since the highest priority at the moment the disaster and during the short time period before the collapse of the buildings is the emergency evacuation of building occupants.

Designing buildings for multiple hazards is also consistent with the objectives of the ongoing National Institute of Standards and Technology (NIST) investigation of the WTC disaster. As NIST Director Arden Bement Jr. stated in August 2002, “The lessons to be learned from this investigation and the companion research and development program are critical to understanding what core reforms are needed to make tall buildings safer nationwide, enhancing the safety of fire and emergency responders, better protecting occupants and property, and providing better emergency response capabilities and procedures for future disasters.” Based on recent findings, NIST is calling for more interaction between structural, fire protection, mechanical, architectural, blast, explosion, earthquake and wind engineering communities; and having fire protection and structural engineers assist emergency personnel in developing plans before entering hazardous buildings.

6.1 Learning from Performance-based Earthquake Engineering

An important model for multi-hazard engineering comes from present practice in “performance-based” earthquake engineering. Traditional earthquake engineering approaches design buildings to withstand a specified earthquake load (the “design earthquake load”); whereas, performance-based engineering aims to meet varied performance criteria for varied hazard intensities.

This approach has been successfully incorporated in seismic design provisions such as FEMA 273 / 356 (FEMA, 1997; FEMA, 2000). According to these FEMA standards, a building is designed to meet discrete safety performance levels: immediate occupancy (s-1), life safety (s-3), and collapse prevention (s-5), each with respect to varied intensities of the earthquake hazard. Intermediate performance levels can also be selected, such as damage control (s-2) and limited safety (s-4). The performance levels are defined to resist different intensities of earthquake ground shaking. Typically, the design criteria are specified for earthquakes at 10%, 5% and 2% probability of exceedance in 50 years. To determine the design criteria for seismic loads, the severity of seismic hazards is measured in terms of amplitude of ground motion (acceleration, velocity and displacement),

frequency contents, and duration of the strong motion segment at the construction site. Factors that may influence the characteristics of ground motion include soil type, fault locations, and distance to the potential sources.

For each safety performance level, earthquake engineers assign building performance objectives to resist the corresponding seismic load. Specific issues considered in performance-based seismic design include: degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, degradation in vertical-load-carrying capacity, falling hazard from structural debris, potential collapse induced by aftershock, and repairability. In view of increasing concerns about homeland security, methods to meet safety performance levels for multiple hazards occurring individually or in combination should be developed.

6.2 Performance-based Multi-hazard Disaster Engineering

The WTC tragedy on September 11 demonstrates that buildings are endangered from multiple nearly simultaneous hazard sources, including collision impact, fire, and explosion. Other dangers to structures include earthquake, flood, hurricane, tornado, winter storms, wildfire, volcano, or other technological and man-made disasters. For future multi-hazard resistant buildings, it will be important to explore methods of performance-based design to extend the approach currently used in earthquake engineering. The development of such a performance-based multi-hazard engineering may need to go through at least five stages as illustrated in Figure 6-1.

At the preparation stage of a multi-hazard performance-based engineering design, an assessment of the occurrence probability of each hazards and their potential combinations should be carried out. Although the scope of work in this stage is beyond the normal assessment carried out for performance-based earthquake engineering, and sometimes the assessment of a potential hazard may even go beyond the probability approach, a quantitative measuring of a possible hazard occurrence is important for setting a reasonable economic constraint in a building design.

At the first stage, hazard conditions are defined by a quantitative classification. Already, many analytical and simplified models, and numerical methods, are available to describe various hazards. The hazard characterization should include measures of hazard intensity, selection of measures that identify the most damaging effects on structures, and measures of factors that exacerbate or attenuate the damaging effects. Table 6-1

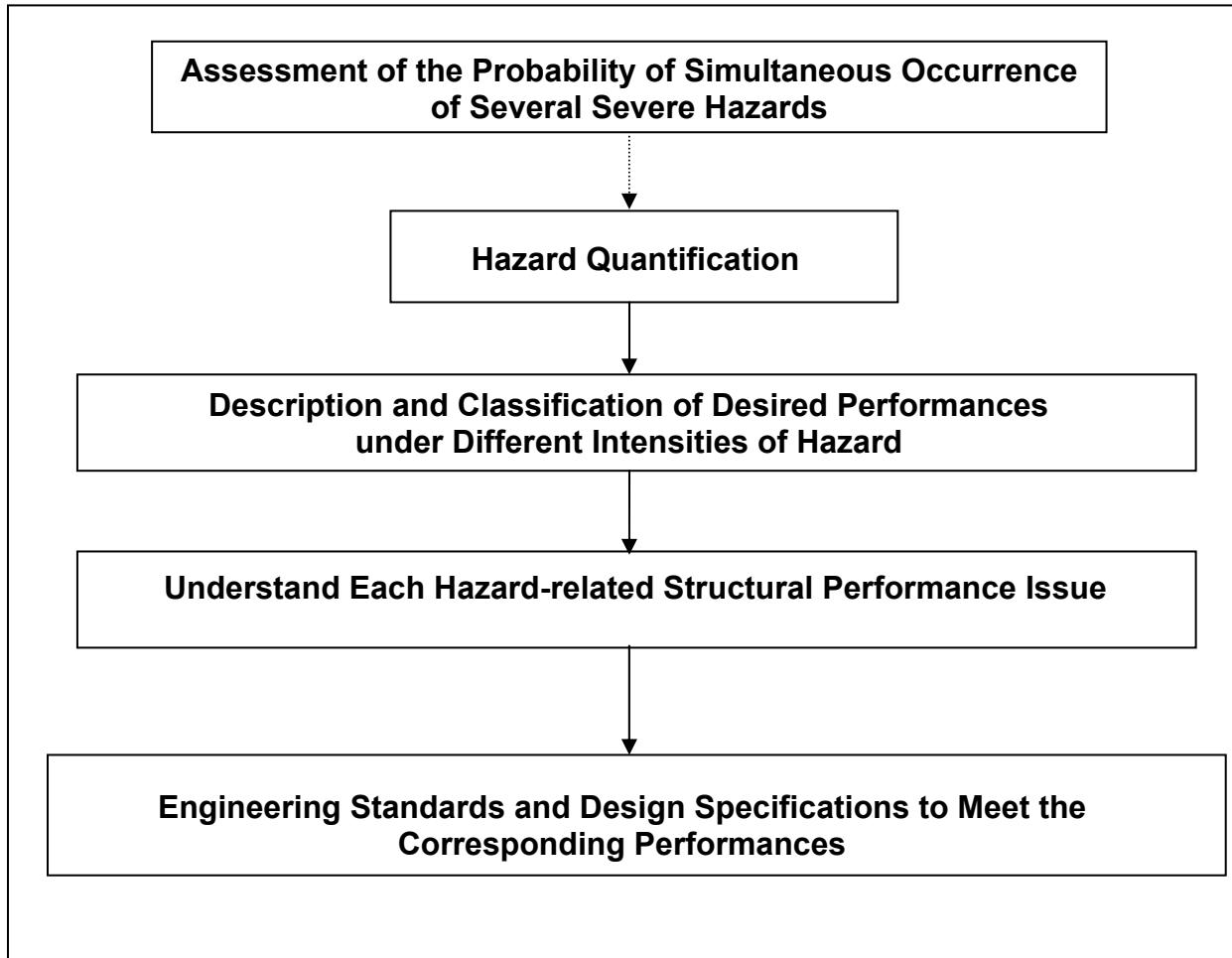


Figure 6-1. Stages of multi-hazard engineering

provides one possible characterization of collision impact, fire, explosion and earthquake hazards.

The second stage is the definition of desired performance levels according to intensities of varied hazards, such as collision impact, fire and explosion. For example, multi-hazard safety standards may consist of (M-1) Immediate Occupancy, (M-3) Life Safety, (M-4) Limited Safety, (M-5), Catastrophe Prevention, corresponding to FEMA s-1 through s-5. However, for extreme events such as the WTC collapse, it may be necessary to add (M-6) Catastrophe Limitation, which does not have a corresponding FEMA s-value. Catastrophe Limitation can include design to delay collapse to give occupants time to escape. Table 6-2 lists potential performance considerations for performance levels M-4 through M-6.

Table 6-1. Potential Characteristics for Impact, Fire and Explosion Hazard/Classification

Hazard Description	Impact Collision	Fire	Explosion	Earthquake
Physical measurement	<ul style="list-style-type: none"> • Force • Acceleration • Energy • Impact area 	<ul style="list-style-type: none"> • Temperature • Heat Energy • Area • Duration of fire 	<ul style="list-style-type: none"> • Pressure • Temperature • Area of contact 	<ul style="list-style-type: none"> • Acceleration • Velocity • Displacement • Energy
Characteristics	<ul style="list-style-type: none"> • Peak level • Duration of impact • Total energy 	<ul style="list-style-type: none"> • Duration of fire • Area of fire • Spatial distribution of temperature • Development of fire 	<ul style="list-style-type: none"> • Spatial distribution of pressure • Duration of high pressure • Peak pressure level 	<ul style="list-style-type: none"> • Peak accel., velocity, and disp. • Total energy • Duration of strong motion
Influential factors	<ul style="list-style-type: none"> • Initial speed • Mass of collision object 	<ul style="list-style-type: none"> • Combustibility of the materials • Ventilation parameter • Fire load density • Distribution of combustibles • Total area of the room's internal surfaces • Thermal inertia of the room's boundaries 	<ul style="list-style-type: none"> • Standoff from detonation point • Amount of explosive materials • Type of explosives 	<ul style="list-style-type: none"> • Source mechanism • Distance to epicenter • Soil and site conditions

The third stage requires research on separate types of hazards to develop structural performance standards. For example, fire-protection design and fire regulations already use the performance-based approach. A typical fire-protection design considers fire initiation, fire expansion and development, magnitude and location of fire, detection of fire and smoke, suppression of fire, egress systems, control of fire growth rate, and other secondary hazards. However, the emphasis of a fire design for buildings is on comprehensive fire hazard prevention. From a structural engineering perspective, what is needed for performance-based structural design in fire hazards is to understand the non-linear behaviors of structural systems under severe fire conditions. In steel structures, potential sources of disastrous failure include thermal expansions, deflection of members caused by degraded material properties, different behaviors of pinned and fixed connections, and internal force redistribution. All the above aspects require further study.

Table 6-2. Considerations for Performance Levels M-4 through M-6

Hazard Performance	Impact	Fire	Explosion
LIMITED SAFETY (M-4)	<ul style="list-style-type: none"> Possible direct injury from objects coming from outside of building. Possible damage to building contents, no major falling objects. 	<ul style="list-style-type: none"> Limited personal injury for persons directly in contact with the fire. Fire constrained in a local area. Only small damage to secondary load-carrying system No damage to major load-carrying system. No major damage to fire protection system 	<ul style="list-style-type: none"> Personal injuries from brittle materials e.g. window glass. No major loss of structural load-carrying capacity. Enough exit routes to evacuate to shelters. Explosion impact contained.
CATASTROPHE PREVENTION (M-5)	<ul style="list-style-type: none"> Damage to major structural members, but sufficient load carrying capability remains. Limited number of personal injuries from direct and indirect impact. No collapse of entire building structure. Major escape routes open. 	<ul style="list-style-type: none"> Limited personal injury. Fire expanded but controlled. Available and well-marked exit routes. Limited damage to major load-carrying system, but sufficient load-carrying capacity. Fire protection system damaged, but still workable. 	<ul style="list-style-type: none"> Local damage to the major structural members won't trigger major collapse. Evacuation/rescue operations. Limited severity of secondary hazards. Temporary sheltering
CATASTROPHE LIMITATION (M-6)	<ul style="list-style-type: none"> Gradual failure of load-carrying capacity to support evacuation. Avoid other severe secondary hazards. 	<ul style="list-style-type: none"> Enough time for escape or rescue. Fire expanded and uncontrollable. Available and well-marked exit routes. Gradual failure of fire resistance. Delayed progressive collapse collapse 	<ul style="list-style-type: none"> Gradual failure of load-carrying capacity to support evacuation. Delay progressive collapse

The fourth stage is the development of integrated analytical tools for determining structural performance across different types of hazards. This stage will require progress in developing commensurable standards across engineering fields concerned with different hazards. Particular attention will have to be paid to conditions when varied hazard types (such as impact, fire, and explosion) affect a structure simultaneously or progressively.

6.3 Research on Structural Performance for Catastrophe Limitation

Most traditional earthquake engineering design has focused on FEMA performance levels s-2 through s-5. However, the levels of hazard conditions for the impact, fire, and explosion from the September 11 attacks all exceeded this standard. Although the kind of disaster that occurred on September 11 is a rare case, it is reasonable to use the event to examine possibilities for higher performance meeting the proposed M-6 Catastrophe Limitation standard. In conditions in which the intensity of the hazard is so great that it is economically impossible to prevent collapse, the purpose of this standard is to achieve a certain limited engineering goal of reducing casualties. This may be achieved by limiting or reducing the rate of collapse (or other catastrophic failure), by providing information on imminent collapse, creating structurally safer shelter within the building, and providing safe passages for escape.

The WTC catastrophe has provided a critical case study toward such catastrophe-limiting design. It does so with respect to each separate hazard (impact, fire, and possibly explosion) as well as the hazards in combination. It appears from the impact analysis that collision alone may have been sufficient to severely undermine the structural integrity of the towers. It especially appears that in addition to the perimeter columns being severely damaged by the airplanes, the high impact force, though not enough to tip over the building, may have brought significant damage to the main truss and column connections. Studying the overall and local collision impact on the progressive collapse of the WTC towers will provide an understanding of the performance expectations under global and local load conditions.

With respect to the fire in the twin towers, many studies have pointed out that these buildings were never considered a likely place for a fire of such size and magnitude. However, the WTC towers can provide data for future performance measures by which to develop fire-resistance measures for worst-case conditions, so that a more appropriate means can be found to reduce the rate of expansion of fire, partially contain fire, provide temporary protection, or improve designs for escape.

6.4 Approaches to Multi-hazard Mitigation

Designing for multiple hazards, rather than for each hazard individually, can result in a greater level of safety and economy. Currently, risk for each hazard is assessed differently, and has qualitatively and quantitatively different performance criteria. Furthermore, treating each individual hazard separately in sequence may result in an unacceptable demand for structural strength, which in reality, may not be necessary. In general, a typical building today does not have a multiple risk management plan nor a design process for all potential hazards. This results in over-expenditure for one or a few hazards and insufficient protection against other potential risks. The key feature in “multi-hazard engineering” is to develop an integrated risk analysis process and performance objectives with practical constraints in mind, so that effective multi-purpose methods and technologies can be employed. This complex and multidisciplinary research challenge deserves serious consideration with high priority.

The following discussion considers the multi-hazard mitigation of progressive collapse of a structure under an extreme loading. Generally the discussion assumes a steel structure, but most of the discussion is applicable to other types of structures as well.

Although the integrity of a structural system can be destroyed by explosion and/or fire, the essential causes of collapse would be various structural failure modes (e.g., structural member or connection failure). An example is the collapse of the Murrah Federal Office Building in 1995. Investigations concluded that the progressive collapse of the building was primarily due to the loss of three major load-carrying columns caused by 4,000 pounds of TNT exploding at a 15-foot standoff. The nine-story Murrah building was an ordinary moment frame structure for which the structural design included wind hazard, but not explosion or earthquake loads. If the structure had been a more seismic-resistant system, such as special moment frame or a dual system as defined in the *NEHRP 2000 Recommended Seismic Provisions for New Buildings*, total damage could have been reduced by 50% to 80% (Corley, 2001). The reason for this expected improvement is that many of the concepts and approaches used in seismic protections are also effective for explosion and fire hazards. For example, shear walls in a dual system will not only increase the lateral resistance to seismic and wind loadings, but would also limit the tendency of air explosion loading to a small area, and help contain fire expansion. The detailing of special moment frames for seismic resistance would increase the survivability of major columns, beams, and connections under extreme load. As this earlier tragedy shows, mitigation measures

meant primarily for one hazard (earthquake) can be applicable to other hazards as well (explosion).

Similar lessons also became apparent from the WTC tragedy. According to the NYC building code for resistance to progressive collapse, the alternative load path design is evaluated by removal of one column or one beam or one panel wall or a corner (two walls). Obviously, the WTC twin towers were more robust than required by this code. However, the extreme loadings in the September 11 scenario far exceeded normal design considerations. To resist or assess the structural system behavior under the extreme loadings from seismic, wind, impact, and explosion, the global integrity and stability of the structure is a priority consideration, and is required by design against all four hazards. Approaches to mitigating the effects of extreme loads can be effective in practice against all four of the hazards. The list below revisits some of the basic structural design approaches from the perspective of designing multi-hazard resilient buildings.

- 1) **Creating alternative load paths:** This approach requires that the structure have an alternative load path for critical load after loss of a major load carrying member. Methods of achieving alternative load paths – redundancy design - are effective against collision impact, explosion, shaking, wind, and structural damage from fire. A major challenge is for alternative load-path design for catastrophic failure under multiple hazard conditions.
- 2) **Increasing capacity for local resistance:** This approach requires that all the critical elements contributing to the stability of the structural system be capable of resisting the intended level of load without failure. Such capability is achieved from material properties and geometrical configurations.
- 3) **Design of connection strength:** In the WTC twin towers, the angle clip joints holding floor joists to the columns on the perimeter wall and the core structure were suspected weak points that probably triggered the progressive collapse. Most likely, the failure of these joints initiated a local floor collapse and introduced pounding forces of 30 times the normal dead load force to the columns in the lower floors, and caused buckling of the columns and collapse of the building with a domino effect. Considering the concentrated impact force from the airplane crash and the pressure loading from the explosion and fire, each of the loadings could have weakened or caused the failure of these joints. Thus, the ultimate limit state of these joints under the combined hazard loadings should be re-examined. Two stages should be analyzed and checked: (1) the state after the crash and the fuel-induced explosion; and (2) the state after the

lasting fire. Each of these investigations can provide lessons for future multi-hazard resistant connection design.

- 4) **Ductility design:** The emphasis in GSA (General Services Administration) guidelines, GSA 2000 (US General Services Administration, 2000), for progressive-collapse analysis and design, is on ductility of key structural elements. This is another area in which an engineering solution to resist one hazard will help to mitigate the risk of other hazards. In the Northridge earthquake, it was found that many fully rigid ductile connections suffered abrupt failure before reaching their expected ductility level. The damage, which started from brittle fractures, eventually resulted in connection-shear failure. Compared with earthquake and wind hazards, collision force and the pressure wave of explosion generally apply extreme dynamic loadings in short duration, that can cause brittle fractures of material with initial imperfections or minor damages more rapidly than can earthquakes. Many connection technologies and designs developed for seismic applications should be applicable and modified for increased capacity.
- 5) **Retrofit of structural weaknesses in existing buildings:** In earthquake engineering, seismic retrofit for existing buildings is one of the challenging areas: structures designed with insufficient seismic resistance typically have “lack of continuity,” which refers to discontinuities in load-bearing capacity. When subjected to extreme loading, structural failures have been identified as directly related to the total base shear, large story drift, and lack of energy dissipation capability. Such an analysis of structural discontinuities in existing buildings conducted by earthquake engineers are also of value in the analysis of vulnerabilities to collision impact, explosion, and fire. The prescriptive requirements developed for seismic retrofit should be a valuable reference for multi-hazard structural design.
- 6) **Building layout design and compartmentalization:** This approach is used to contain fire or explosion, protect sections of the building from fire or explosion, and/or design critical paths for evacuation. Whereas some of the approaches above were meant to increase the structure’s ability to resist loads, an equally important design approach is to reduce the structure’s exposure to the load. It is sometimes possible to do so through compartmentalization. With properly designed and compartmented zones, fire- and explosion-resistant walls can be used to contain design hazards. Although, in view of the September 11 disaster, containment might not prevent a building from collapse, at least some evacuation routes may be secured.

-
- 7) **Special geometrical shape design:** The configuration of building frame systems may provide another way to reduce the impact of explosion or attack. Although such a consideration has not been fully explored, it is worthy of pursuit.
 - 8) **Sensor networks:** Sensing is one of the technological fields that can be a significant benefit in multi-hazard warning, disaster assessment, monitoring, decision-support systems, and detection of structural damage. Although sensors are not currently part of structural design, they are essential to provide valuable information for real time damage assessment and post-disaster investigation and analysis. If the two WTC towers had an integrated fire, temperature, smoke and structural member condition monitoring and assessment sensing system, the fire-fighting, rescue and evacuation operation would have been better enabled so that the lives of many rescue personnel could have been saved. In the future, sensors will be integrated with structural elements as the pursuit of "intelligent buildings" reaches a certain level of maturity.
 - 9) **Preventive technologies:** Energy dissipation technologies such as dampers, shock absorbers, and fire resistant and separation devices can benefit multi-hazard mitigation. Damper technologies have been successfully used for earthquake and wind engineering in buildings. In addition to contributing to the ductility of the structural system, dampers provide additional energy dissipation capability. However, the effectiveness of dampers will depend on their design load, structural dynamic responses, and the working environment. For example, their performance under high temperature or airwave pressure needs to be established. Modifications to these technologies should be explored for potential explosion protection of structural systems and major structural joints and members in addition to earthquake response reduction.
 - 10) **High performance and composite materials:** One of the fastest advancing fields in engineering is the development of high performance and composite materials. High strength materials provide cost-effective engineering solutions for building protection. For steel structures, high temperature resistant steel and fire protection painting materials with advanced performance are being developed. Concrete wrapped steel columns and other hybrid structural constructions are often used for dual load-carrying and fire-resistant purposes. Other high strength carbon fiber composite materials or glass fiber enhanced construction materials have demonstrated high strength and high hazard resistant capability. These materials are candidates for multi-purpose hazard control applications.

6.5 Looking into the Future: Building Design for Multiple Hazards

In addition to the basic engineering approaches described in the previous section, future building design for multiple hazards should consider a broader scope of issues. As discussed earlier, the lessons learned from the severe multi-hazard loading to a building structure are extensive and beyond current design considerations. The collapse of a building under such loading conditions is almost inevitable. However, the initial design never considered how the building collapse process develops. Issues related to this process are open research topics. Perhaps, the design philosophy of “collapse prevention vs. life saving” should be revisited and included in the development of design approaches for severe multi-loading conditions. Further, the performance goal for a building under such conditions should be changed to “How should buildings be designed so that collapse may be adequately controlled or sufficiently delayed to save lives?” and “How should buildings be designed to help the search and rescue tasks if the building is collapsed?” A multi-hazard resilient building design should certainly include such a performance consideration in combination with redundancy, alternate load-transfer paths, use of response control technologies, and so on.

While it may not be technically possible or fiscally justifiable to design and construct landmark buildings against similar jet airplane attacks and subsequent collapse, it may be feasible to explore mitigation measures and methods (to delay collapse), together with emergency response and management to limit the loss and further disaster development, should a similar disaster strike in the future. Under this consideration, Figure 6-2 illustrates a comprehensive framework for a multi-hazard resilient system that integrates the various issues and problem phases in mitigation for landmark buildings and critical facilities. When a hazard or a combination of several hazards strikes a facility, the occupants, operational systems, utility systems and structural systems are subjected to a complex assortment of impacts. Response and recovery activities will depend on the severity of the impact and the function of the facility. When damage to the facility reaches a certain level, the objectives of response and recovery usually change from hazard control to disaster relief. Aftermath studies typically can help to illuminate some weaknesses and defects in existing hazard prevention and mitigation measures. Then, using this information, performance objectives can be developed or improved. In post-disaster reconnaissance, the next generation of risk assessment, mitigation, response and preparedness planning are beginning to be implemented. In particular, new technologies and engineering solutions are expected to meet the technical demand set forth based on improved performance objectives. Information about hazards and conditions, vulnerability of the facility, and possible needs for disaster

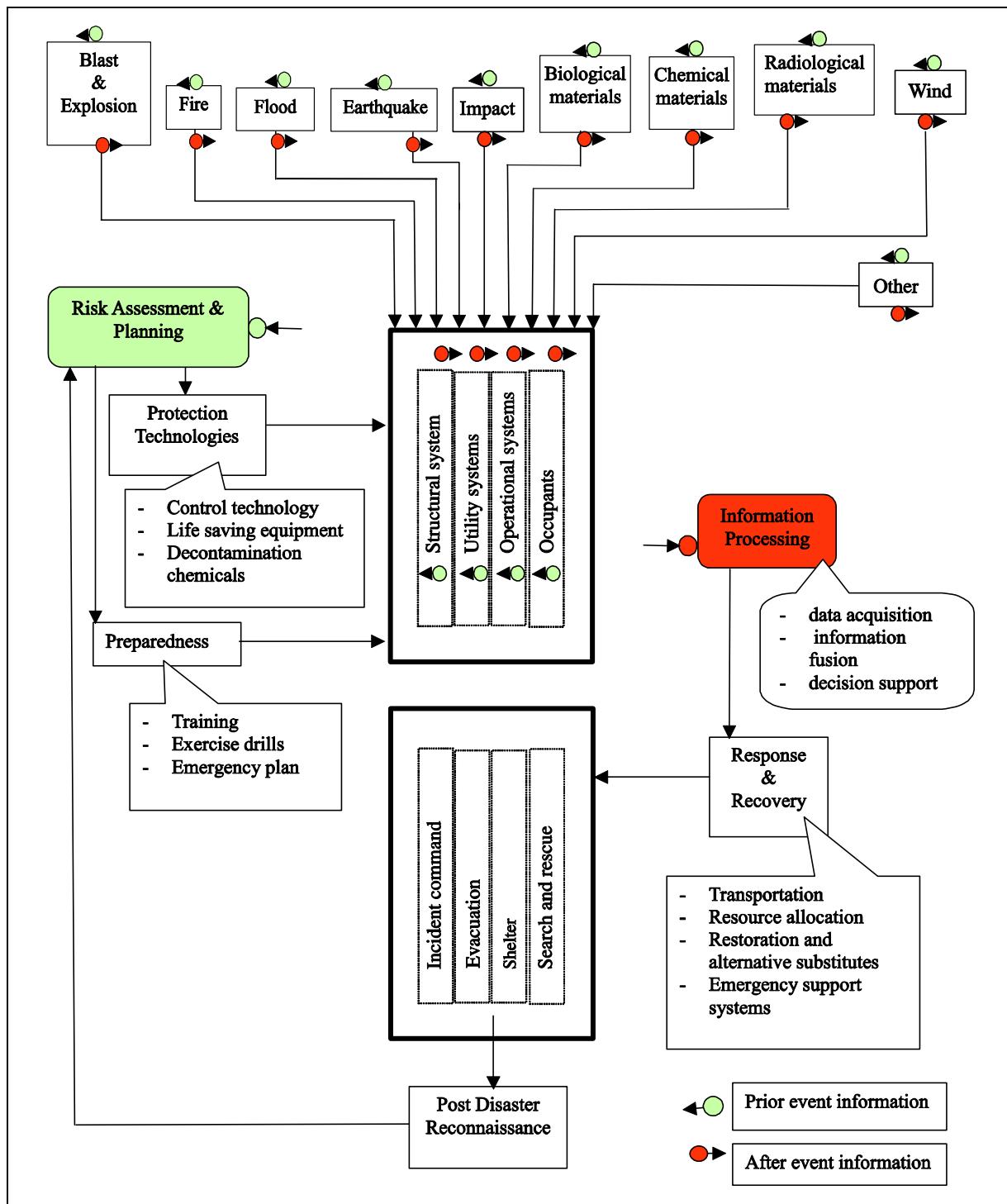


Figure 6-2. Integrated multi-hazard resilient system

response, shown in the figure by a light circle and arrow, is considered in the next pre-event planning and engineering design cycle. In contrast, during the disaster, information such as hazard intensity, condition of facility damage, impact to occupants, etc., indicated by the dark circle and arrows, can be used for real-time incident decision support.

In the end, the lessons learned from this World Trade Center tragedy will encompass much more than the first level considerations of improved design, standards and codes for buildings and other infrastructure. At the higher level, we have learned more lessons regarding the efficiencies of organizational communication and coordination and the utilization of advanced technologies in disaster mitigation management and response. The memorial to be built at the WTC site in lower Manhattan will provide a lasting reminder for future generations to help them understand the magnitude of what happened on September 11, 2001. In this context, while the engineering profession holds the key to understanding the technical impact of such events, and cooperation among institutional systems can significantly increase the disaster resiliency of the community, those in the humanities deeply understand its impact on the hearts and minds of people. The extraordinary expression of art, music, literature, philosophy and history, such as the proposed memorial, can serve to change "disaster preparedness" from responding to a disaster with a sudden demand of money and manpower to a regular budgeting and educational activity, which should be more cost effective with less disruption to the physical, organizational and human systems of our society in the long run. Therefore, the highest level of lessons learned is how to integrate the humanities perspective into the engineering and organizational perspective, into a new field of "multi-hazard disaster engineering."

These "disaster engineers" of the future will have a more complete "toolbox" to draw from when or if a large and unexpected disaster occurs. Instead of responding to a specific hazard, natural or man-made, they will have already taken steps and gained the knowledge needed to better ensure the safety of people should such an event occur. By developing an overall plan to mitigate against a wide variety of hazards, limited dollars can be more wisely spent to provide protection against these hazards. In fact, mitigation becomes part of the life-cycle cost of a building or other infrastructure, rather than retrofitting to protect against a specific hazard. This will require a long and sustained effort in education, training and policy changes. We strongly advocate the development of education programs to train future professionals to be "disaster engineers." These programs may be in the form of short courses for working professionals, course components in existing educational programs, or a Master of Engineering degree program dedicated to the development of disaster engineering professionals.

7.0 Summary

This report began by summarizing the authors understanding of the collapse of the WTC towers from a structural engineering perspective to identify similarities and dissimilarities between blast resistant design and earthquake resistant design of buildings. There have been numerous studies reporting various possible failure modes/mechanisms that led to the collapses of the WTC towers. These studies principally focus on airplane impact and elevated temperature due to fire. There are many “theories” for the collapse of the buildings and many “lessons learned” for future structural design practice against the devastating loading conditions experienced by the WTC towers.

The authors have concluded that the “major” lesson learned from the WTC disaster is the need for a more concerted and integrated effort in research, education and professional capacity of “multi-hazard disaster engineering” in our society, particularly with respect to the hazard protection and response management of physical facilities and infrastructure systems. This calls for a sustained, multidisciplinary team effort.

Following the discussion on important structural design lessons learned from the collapse of the WTC towers, this report proposes a framework based on performance-based earthquake engineering to address severe multi-hazard loadings of critical buildings. Because the demand on building performance will change instantly under unexpected destructive loads such as the ones experienced by the WTC towers, the highest priority may be emergency evacuation of building occupants. The idea of “controlled” or delayed collapse should be considered for heavily occupied buildings.

In general, hazards do not occur simultaneously. Their consideration during the building design phase can be more cost-effective, not only in construction but more importantly, for long term maintenance purposes as well. To realize such benefits, properly educated professionals are necessary. This report strongly advocates the development of a professional engineering degree program or training courses in disaster engineering.

Since its inception in 1986, MCEER (then NCEER) has sought to integrate engineering disciplines (geotechnical, structural, mechanical, material science, etc.) to better mitigate the impact of earthquakes. This evolved to the development of the current MCEER vision to establish earthquake resilient communities to include relevant social science components, and at the same time, to educate engineering professionals with multidisciplinary capability. The next challenge is to develop these combined engineering/social

components into a much bigger, and more relevant social and cultural context. These new professionals can carry on the continued pursuit to change our practice in dealing with natural and manmade disaster from rapid response to the unexpected to an integrated approach encompassing multi-hazard disaster preparedness with life cycle maintenance of physical facilities, which ultimately will be less disruptive to our human society.

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Appendix A

Structural System of the World Trade Center Towers

The structural system of the WTC towers is briefly reviewed to provide the basis for describing the airplane collision and subsequent fire and explosion, and their possible roles in the structural damage and collapse of the buildings.

In the analysis carried out in this report, the structural data were mainly taken from FEMA (2002). In addition, some relevant information from the web site "World Trade Center and Pentagon Tragedy" at <http://www.engr.psu.edu/ae/wtc/wtctragedy.html> is also used, which may not be available now (Geschwindner, 2003).

The dimensions of the WTC towers are summarized in Table A-1.

Table A-1. Dimensions of the WTC Towers

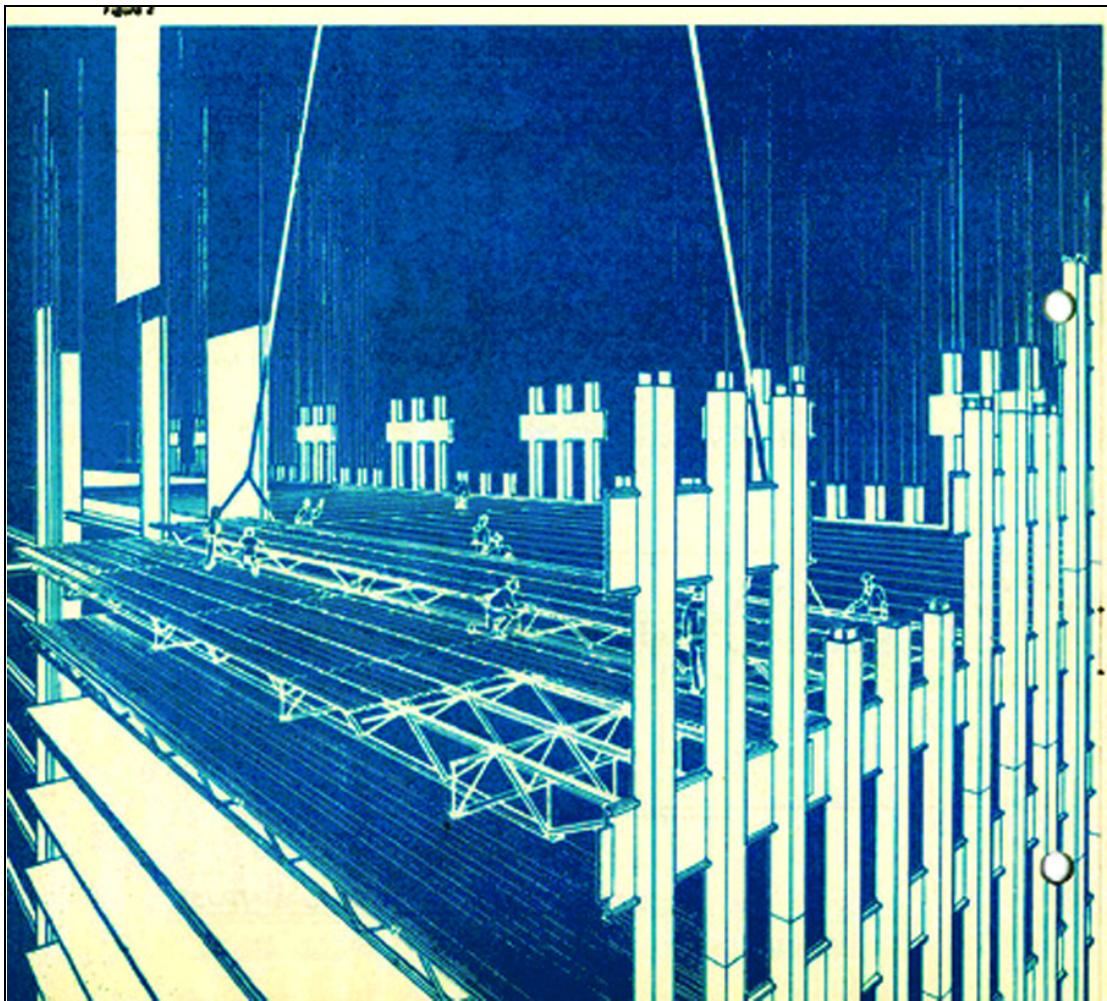
	North Tower (WTC1)	South Tower (WTC2)
Height	1368 ft (417m)	1362 ft (415m)
Floor space	207 ft X 207 ft (63m X 63m)	207 ft X 207 ft (63m X 63m)
Central core	87 ft X 137 ft (26.52m X 41.76m)	87 ft X 137 ft. (26.52 X 41.76m)
Floor Height	12 ft (3.66m)	12 ft (3.66m)

The central core frames were oriented to the E-W direction in WTC1, and the N-S direction in WTC2.

The load bearing structural system of the WTC towers consisted of external bearing walls on the perimeter of the buildings, the central core frames, and floor supporting trusses. Figure A-1 (Rapp, 1964) is a three-dimensional segment view of external bearing wall frames and floor supporting trusses. Figure A-2 (Rapp, 1964) is a design drawing with dimensional information of the panels that formed the external walls. Figure A-3 (FEMA, 2002) shows the sizes of the columns and spandrel beams for the external bearing walls.

The external walls consist of steel box columns and internal beams (spandrel plates). The box columns were formed by welding four plates to a square

section approximately 14 in. x 14 in. (The actual dimension of the section is shown in Fig. A-3.) At each floor level, spandrel plates were interconnected to the perimeter columns. A typical dimension of a spandrel plate is 54 in. x 3/8 in.



Rapp, 1964

Figure A-1. Three-dimensional view of external wall frames and floor trusses

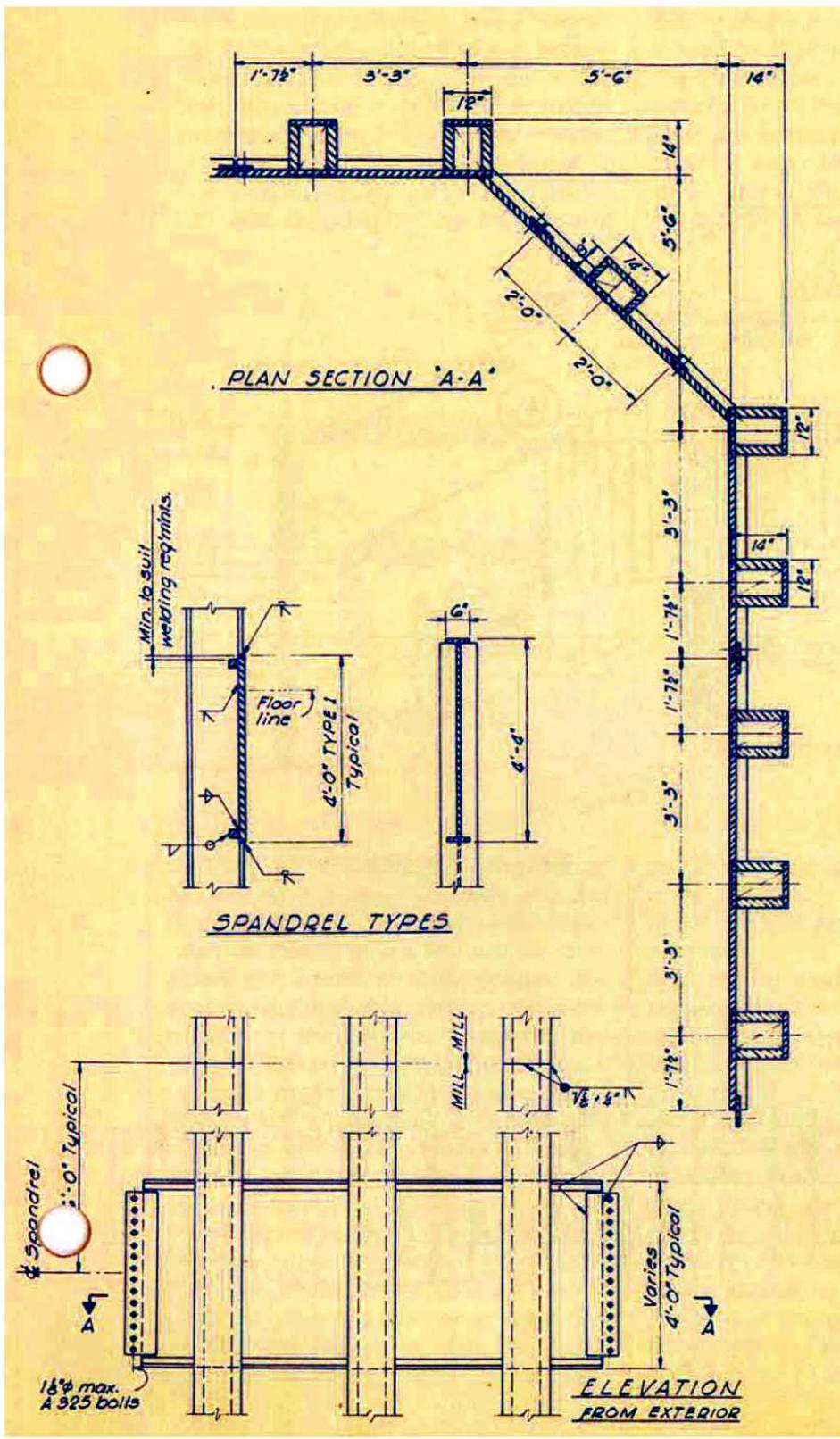
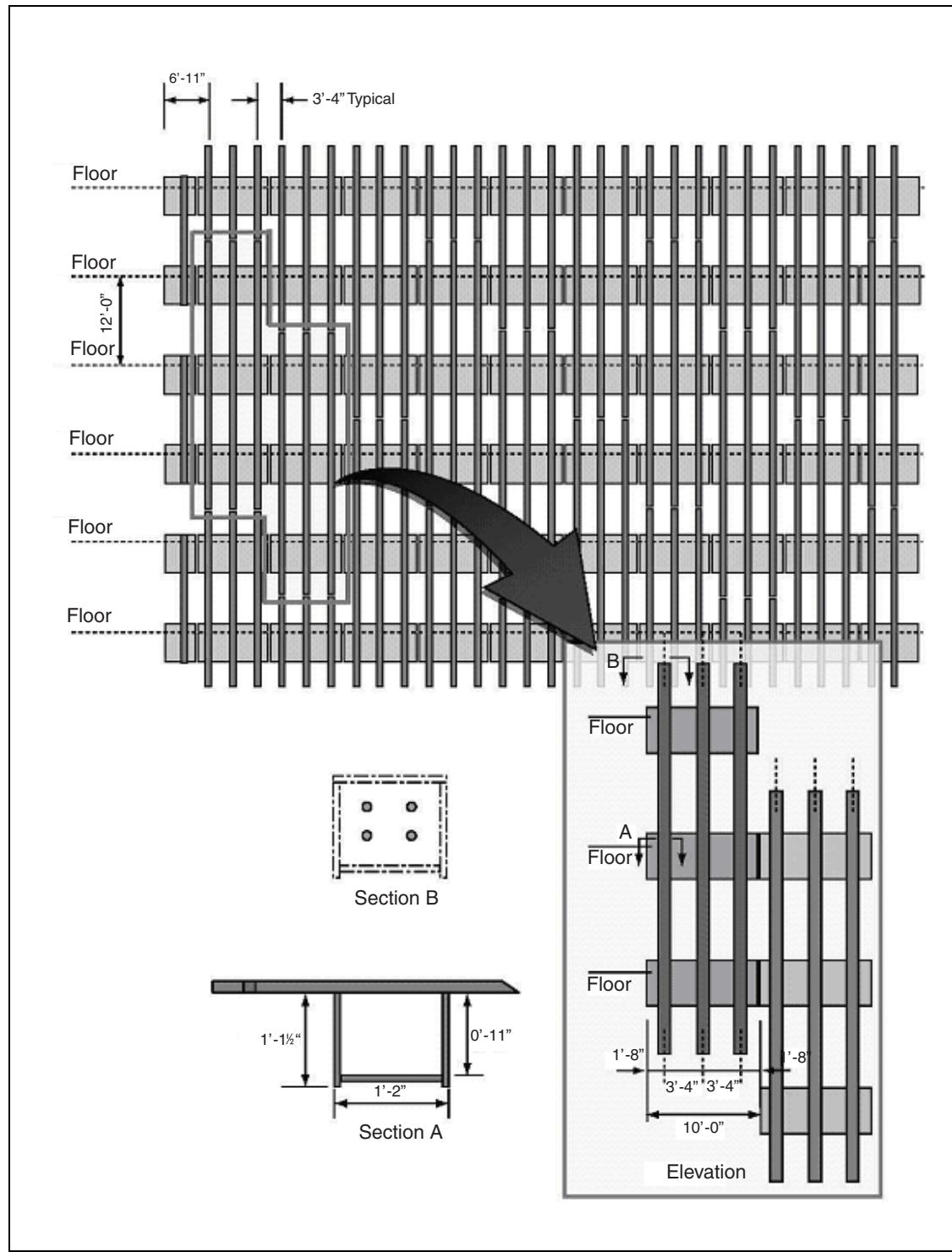


Figure A-2. Exterior wall sections

Rapp, 1964



FEMA, 2002

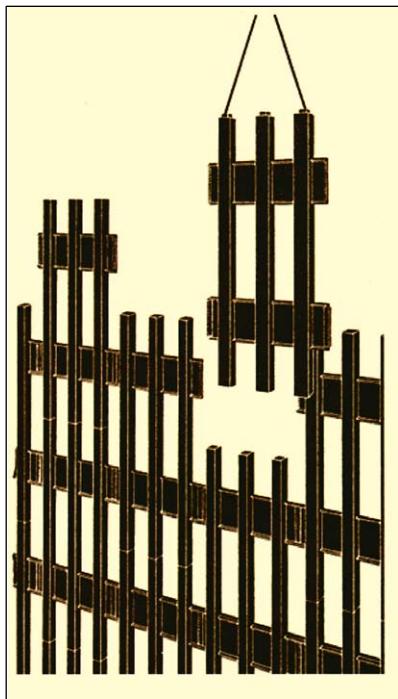
Figure A-3. External bearing-wall frame system

The perimeter walls were constructed from modules, each three stories high and three bays wide (36 ft x 10 ft). Modules were connected by high-strength bolts at the middle two floors in the vertical direction and at mid-span in the horizontal direction. A four-bolt connection plate was used for upper levels of the building; a six-bolt connection plate for lower levels.

The modules were assembled with staggered joints along the height so that less than one-third were split between any single story. Figures A-4 and A-5 are views of the module at assembly. Note that in the bottom seven stories, regular three-column modules were replaced by single-column modules, as shown in Figure A-6. Figure A-7 is a view of the World Trade Center during construction.

Various grades of steel were used for the towers: all columns in the central core area were A36 steel. For the exterior columns, 12 different grades were used. The yield strength for these steel members varied from 36 ksi (A36) to 100 ksi (A514).

Sections of columns and beams varied along the height of the building. In the upper stories, a $\frac{1}{4}$ inch thick steel plate was used for the columns; and a $\frac{3}{8}$ inch steel plate was used for the spandrel beams.



Rapp, 1964



FEMA, 2002

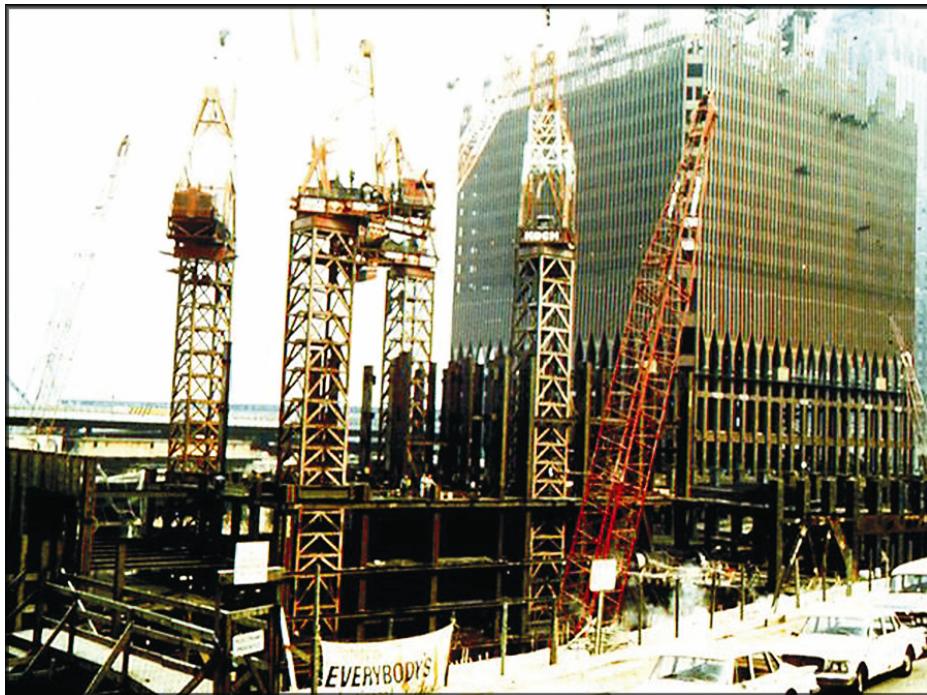
Figure A-4. Prefabricated modules of exterior walls

Figure A-5. Prefabricated modules of exterior walls



Geschwindner, 2003

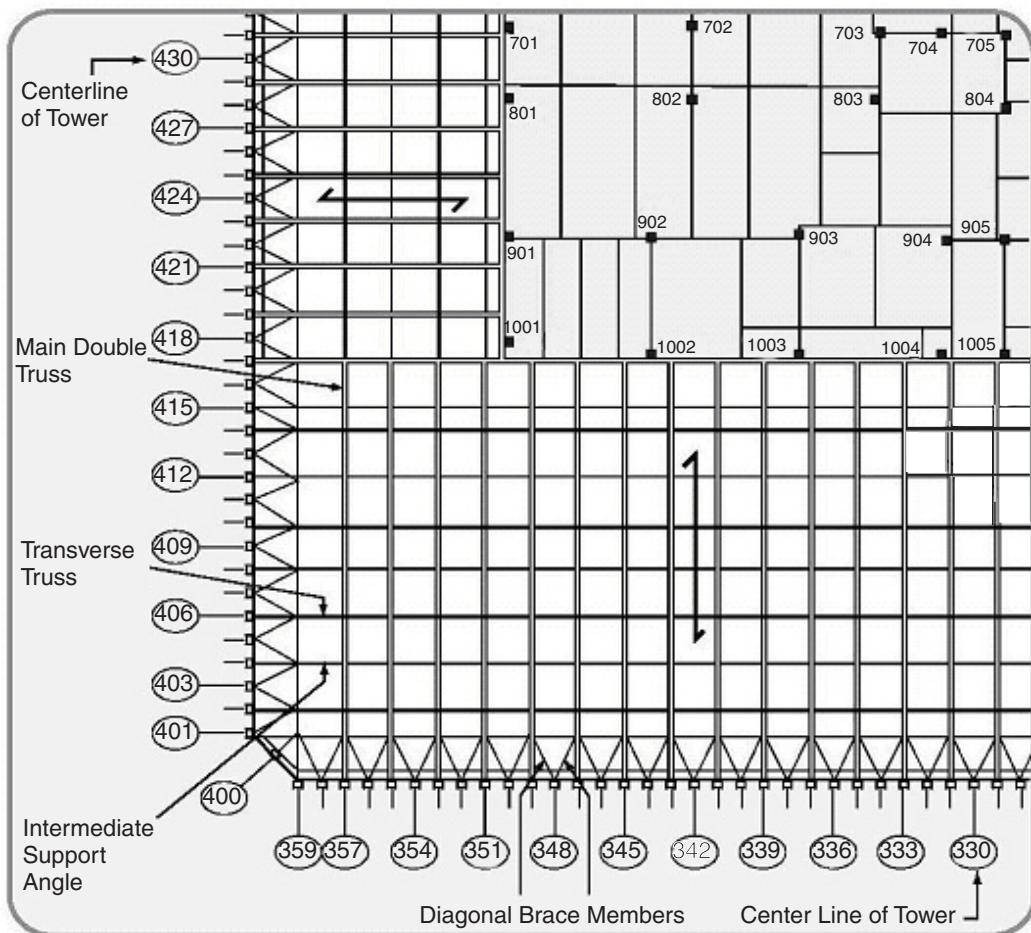
Figure A-6. WTC in the process of construction



Geschwindner, 2003

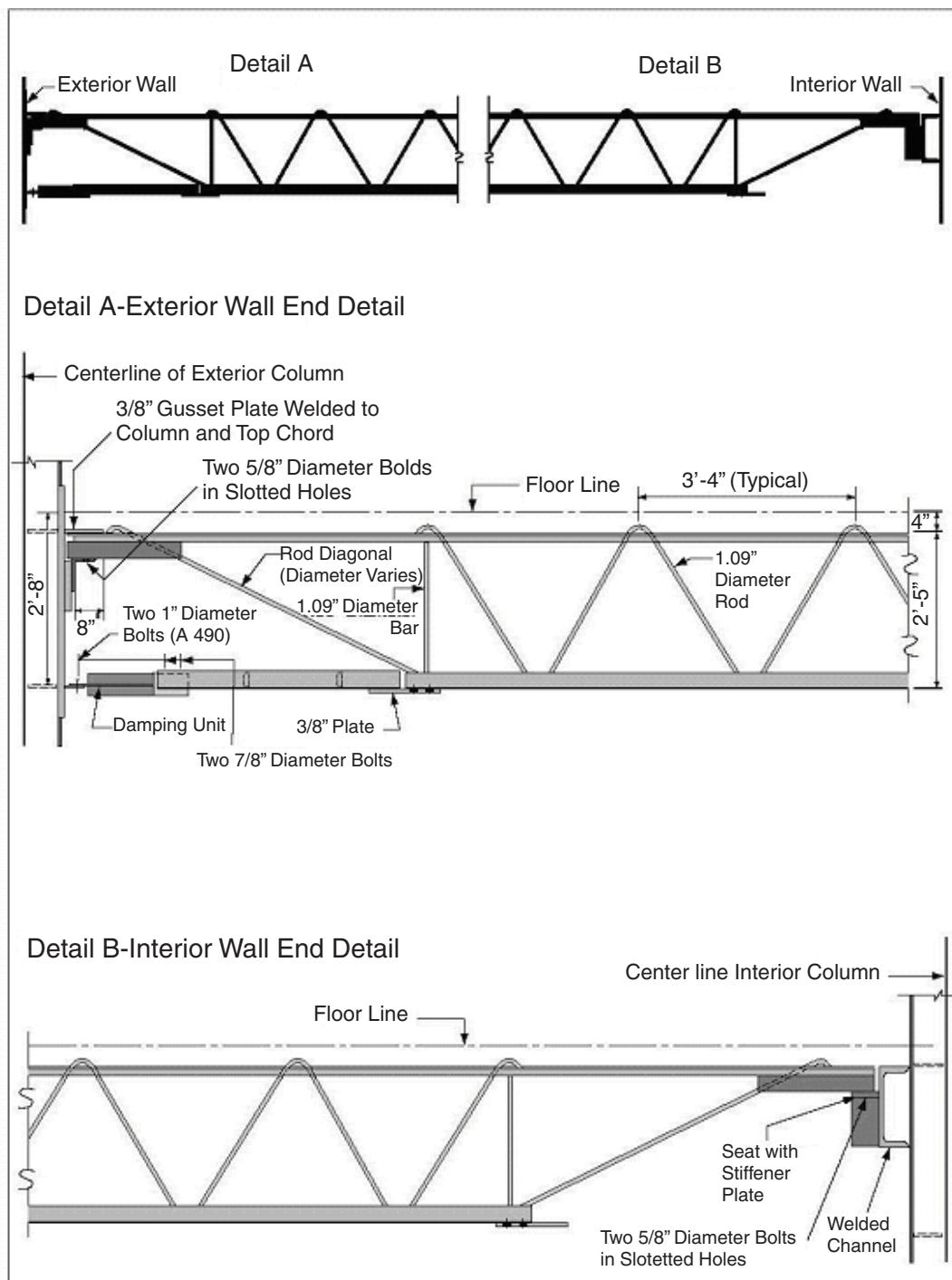
Figure A-7. WTC in the process of construction

The floor load bearing system consisted of steel trusses of 60 ft. and 35 ft. spans in the two perpendicular directions. These trusses were connected to the exterior wall frames and beams of the central core system by bolted and welded connections. Lightweight concrete slabs 4 to 5 inches thick were placed on top of the steel decks. A plan view of a typical floor segment with the supporting trusses is shown in Figures A-8 and A-9 show some details of floor trusses and connections. Fig. A-10 shows the construction of the floor.



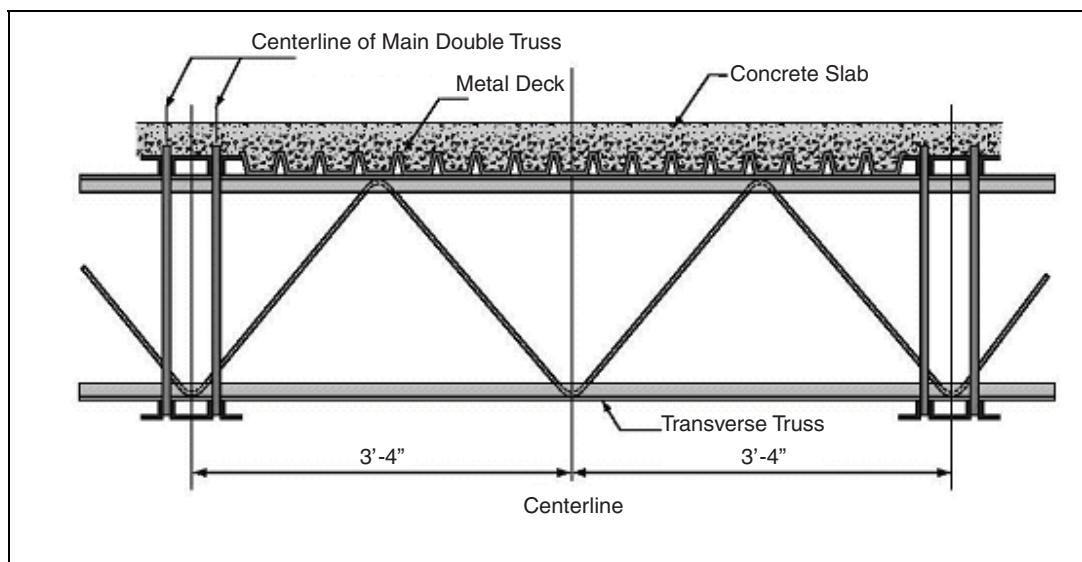
FEMA, 2002

*Figure A-8. Floor plan with core columns and trusses distribution
(1/4 part of the floor)*



FEMA, 2002

Figure A-9. Trusses and details on the typical floor



FEMA, 2002

Figure A-10. Floor construction

Appendix B

Temperature-dependent Mechanical and Thermal Properties of Structural Steel

B.1 Thermal Properties

Steel will expand at elevated temperatures. The thermal expansion coefficient of steel adopted by AISC is given by equation (B-1):

$$\alpha_s = (11 + 0.062T_s) \times 10^{-6} \quad \text{m}/(\text{m} \cdot ^\circ\text{C}) \quad T_s \leq 600^\circ\text{C} \quad (\text{B-1})$$

According to the experimental studies of Cooke (1988), the thermal expansion coefficient does not always increase as the temperature increases. Cooke has shown that the expansion coefficient for steel will decrease at approximately 800° C (1,500° F).

For the WTC towers, when the temperature of the steel members reached 600~700° C (1,100~1,300° F), the thermal expansion coefficient should be in the range of $48.2 \times 10^{-6} \sim 54.4 \times 10^{-6}$ m/(m·°C). For the temperature of 300 ° C (570° F), it should be 29.6×10^{-6} m/(m· °C).

Gillie, Usmani, and Rotter (2001, 2002) have studied structural response due to fire in one of a series of tests performed on a full-scale, eight-story, steel-framed building in 1995 at Cardington, UK. Their results indicate that the response of the structure is overwhelmingly dominated by the effects of thermal expansion rather than material degradation.

Thermal conductivity is another important property for hazard considerations of steel structures subjected to elevated temperature. For carbon steels, it usually varies within the range of 46 to 65 W·m⁻¹·K⁻¹.

B.2 Mechanical Properties

There are many fire-related mechanical properties of structural steel. In this section, only the strength and modulus of elasticity at elevated temperature and their effects on the collapse of the WTC towers are briefly discussed.

Strength

Figure B-1 shows the stress-strain relationship at room temperature and elevated temperatures for structural steel (ASTM A36). Figure B-2 shows the effects of temperature on the yield and ultimate strengths of A36 and A421. Note that the yield strength is reduced by approximately 50% at 550° C (1,022° F). A mathematical expression for this reduction is given by equation (B-2) (Milke, 2002):

$$\sigma_{yt} = \begin{cases} \left(1 + \frac{T}{900 \ln(T/1750)}\right) \sigma_{yo} & \text{for } 0 < T \leq 600^\circ \text{C} \\ \frac{340 - 0.34T}{T - 240} \sigma_{yo} & \text{for } T > 600^\circ \text{C} \end{cases} \quad (\text{B-2})$$

where σ_{yt} is the yield strength at temperature T (MPa), σ_{yo} is the yield strength at 20° C (68° F) (MPa), and T is the steel temperature.

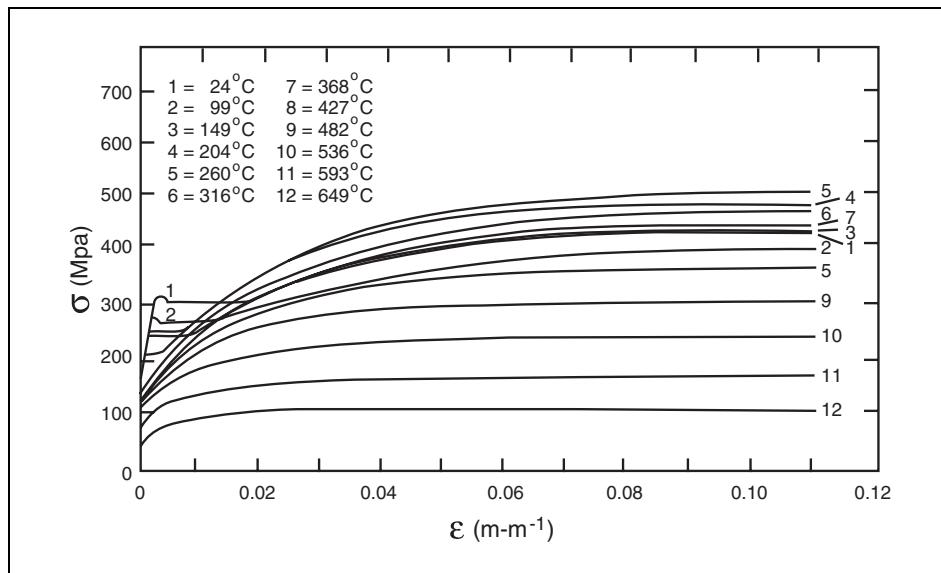
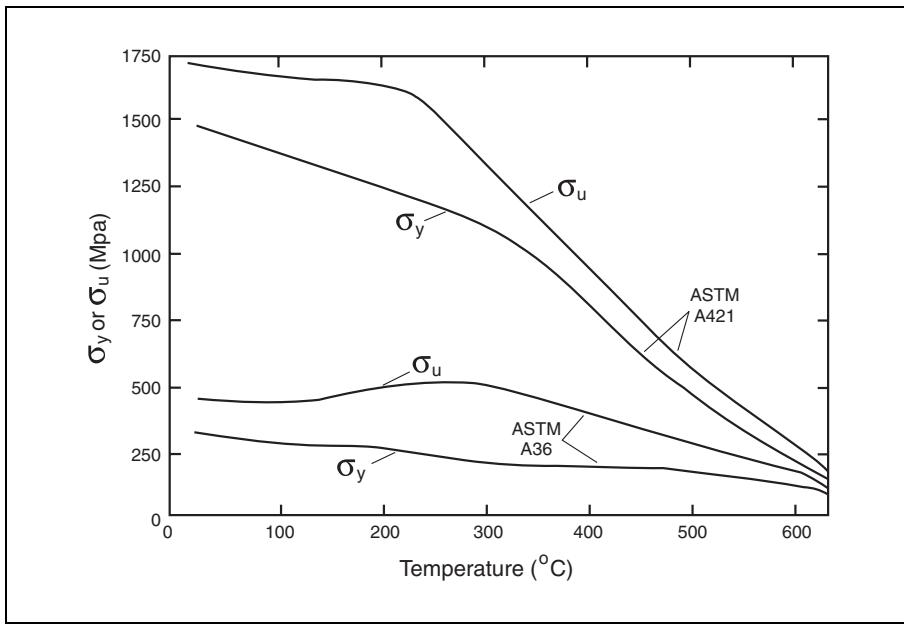


Figure B-1. Stress-strain curves for A36 at room temperature and elevated temperatures



Kodur and Harnathy, 2002

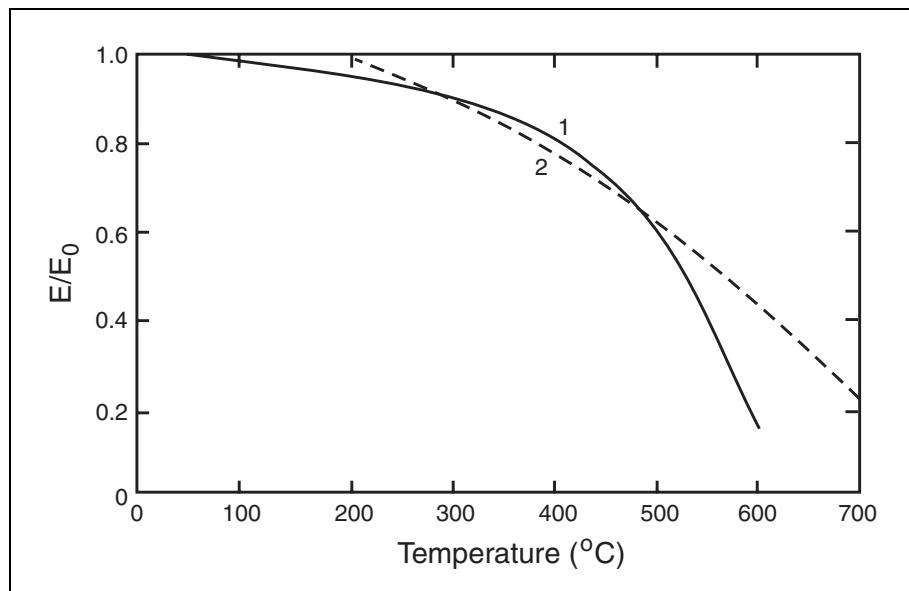
Figure B-2. Ultimate and yield strength for A36 and A421 (prestressing steel) at elevated temperatures

Modulus of Elasticity

Figure B-3 presents the variation of the modulus of elasticity as a function of temperature for structural steel and steel reinforcing bars according to SFPE 2002. When the temperature reaches about 550° C (1,022° F), the modulus of elasticity is reduced to approximately half of its value at room temperature (20° C), which is about 210×10^3 MPa. The mathematical expression is given by (Milke, 2002):

$$E_T = \begin{cases} \left(1 + \frac{T}{2000 \ln(T/1100)}\right) E_0 & \text{for } 0 < T \leq 600^\circ \text{C} \\ \frac{690 - 0.69T}{T - 53.5} E_0 & \text{for } T > 600^\circ \text{C} \end{cases} \quad (\text{B-3})$$

where E_T is the modulus of elasticity at temperature T (MPa) and E_0 is the modulus of elasticity at 20° C (68° F) (MPa).



Kodur and Harnathy, 2002

Figure B-3. Modulus of elasticity at elevated temperatures for structural steels and steel reinforcing bar

Mechanical Properties of the WTC Towers in Fire

For the WTC towers, the steel members in the immediate area of collision may have reached about 600~700° C (1,100~1,300° F). The yield strength of these members was reduced to only 22%~38% of the value at 20° C (68° F) and modulus of elasticity was reduced to 32%~50% of the value at 20° C (68° F). The reduction of strength and modulus of elasticity certainly affected the load carrying capacities of the structural members, which contributed to the eventual collapse of the building.

At a temperature of 300° C (570° F), the steel members still have 81% of the strength and 88% of the modulus of elasticity at 20° C (68° F). Analysis shows that the WTC towers could survive with only 50% of the strength and the modulus of elasticity, if no additional loadings were applied to the structure. The above estimates may help to explain why the towers remained standing for 56 minutes and 102 minutes, respectively, after the collisions.

B.3 Critical Temperature for Various Types of Steel Members

The critical temperature is defined as the temperature at which the material loses much of its strength and can no longer support the applied load. The critical temperatures for various members are shown in Table B.1. These values will increase when the stress level due to applied loads are reduced.

Table B-1. Critical temperatures for various types of steel

Type of steel member	Temperature
Columns	538°C (1,000°F)
Beams	593°C (1,100°F)
Open web steel joist	593°C (1,100 °F)
Reinforcing steel	593°C (1,100°F)
Prestressing steel	426°C (800°F)

FEMA, 2002

Appendix C

Background Information and Models of VCE

Fire hazards caused by liquid fuel spillovers are generally classified as flash fires and vapor cloud explosions. When contained liquid fuel is released, a fuel vapor/air mixed cloud is generated. Depending on the fuel density in the mixture and the temperature and pressure of the environment, the ignited cloud could be either a flash fire (non-explosive combustion) or a vapor cloud explosion (VCE) (explosive combustion).

Flash fires and vapor cloud explosions have two different combustion modes. When a large amount of fuel vapor exists unmixed with sufficient air, the cloud's ignition results in a flash fire and the rapid expansion of a fireball. When a large amount of fuel vapor mixes with sufficient air to form a large cloud before fire ignition, a vapor cloud explosion (VCE) may occur. A VCE has two possible explosive combustion modes: deflagration or detonation. The former has subsonic flame propagation; and the latter is associated with supersonic flame propagation. It has been observed that turbulence always develops with VCE, which pushes the propagating flame ahead, causing continued expansion. The overpressure wave front is a typical damaging characteristic of this type of VCE.

For a large tank of fuel mixed with a certain amount of air, a fire could result in a major explosion. Sometimes, the combustion (and explosion) of the hydrocarbon fuel can develop into a deflagration fire. In severe cases, it may become a detonation, which is often associated with a "triggering mechanism," such as a TNT explosion.

The most important parameter for structural design against VCE loading is the overpressure in front of the explosion wave, which is often difficult to estimate.

In this study, based on incomplete information, the detonation process is examined since it is the worst case scenario. The VCE of deflagration process, not pursued in detail, is also briefly explained.

The visible spreading speed of a normal fire flame (non-explosion mode) under free-air conditions is less than 10 – 15 m/sec. The leading and trailing pressures of the flame do not exceed 0.1 kg/m^2 . For a fire in a contained space, with full combustion, the pressure could reach 60 – 120 T/m^2 . In

deflagration, the flame front typically spreads at less than sonic speed, and the speed and temperature of the flame are related to the characteristics of the gas-air mixture (vapor cloud). For deflagration (sub-sonic explosion mode), the spreading speed of burning could reach 30 m/sec, and the pressure doesn't normally exceed 0.2 – 0.5 kg/m². For detonation (super-sonic explosion mode), the cloud zone is generated at the initial moment of explosion. The size of a cloud zone depends on the mass and the characteristics of the gas-air mixture. After explosion, the front of the shock wave will quickly reach the boundary between the VC zone and the non-pressurized air zone to begin a process of "explosion produced dilatation" (enlargement). The enlargement of gas-air mixture explosion is 5 – 6 times the initial radius of the VC zone. For a TNT explosion, the enlargement is 800 – 1600 times of the size of the explosion source, but the initial radius of VCE is much larger than the radius of the TNT charge because of the difference in the density of the gas-air mixture and that of TNT.

Several theories are available for predicting the characteristic parameters of VCE (see Strelchuk and Orlov (1981)). Four vapor cloud explosion models are commonly used. The similarities, differences and capacities of these models are discussed in *The Quest Quarter*, Spring 1999 and Summer 1999. It is concluded that these models adequately predict the pressure for distances beyond 100 m from the center of explosion, but their accuracy near the vicinity of the center of an explosion is questionable.

The zone close to the center of explosion (0 - 100 feet) is the focus of this study. The models of Bodansky et al. (1974) and Strelchuk and Orlov (1981) are used because they are more suitable for close range prediction of the pressure. The model of Bodansky et al. (1974) can be used only for the detonation process.

In subsequent discussions of the explosions in the WTC towers, the Strelchuk and Orlov model is used for the deflagration processes and the model of Bodansky et al. is used for the detonation processes.

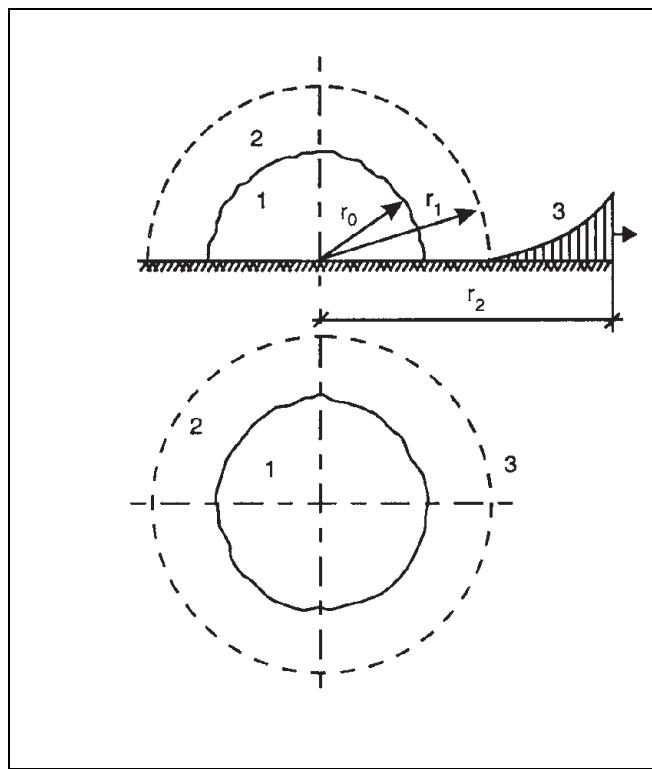
C.1 VCE Deflagration Process

In the deflagration process, the explosion pressure applied to the surface of a structure depends on several factors: (1) ratio of the volume of burning fuel, V_b , to the volume of the enclosed space, V_p ($m = V_b / V_p$); (2) properties of the burning substances; (3) zone of burning fire (normal zone or accelerating zone); and (4) burning conditions (contained or open space). Strelchuk and Orlov developed overpressure curves as function of m ratio and the other three parameters.

Consider that the quantity of jet fuel carried by the two airplanes was approximately 10,000 gallons and the dimensions of the WTC floor area over the story height, V_b / V_p , was in the range of 0.001 - 0.005. According to the overpressure curves by Strelchuk and Orlov, in an accelerating zone of burning with the fastest flame expansion rate, the overpressure would be in the range of 0.05 - 0.1 kg/cm² ($\approx 0.7 - 1.5$ psi). Such overpressure magnitude could cause injury to people and breakage of glass windows, but damage to structural frames and load bearing walls system would not be likely. Since deflagration is a much less severe explosion, its main effect is similar to that of the fire, which are high temperature-induced material property degradation and heat energy released from the fire. The pressure associated with the deflagration is not pursued further in this report.

C.2 VCE Detonation Process

The VCE detonation process can be generally subdivided into three conditional zones: (1) the vapor cloud zone (radius r_0); (2) the explosion escalation zone (radius r_1); and (3) the air shock wave zone (radius r_2). Figure C-1 shows the three zones of a VCE explosion.



Bodansky et al., 1974

Figure C-1. Ground gas-air mixture explosion

The main parameters of this type of VCE explosion and their relationships are summarized in this Appendix. They are used for the estimation of VCE damages of the WTC towers reported in Chapter 4 (Bodansky et al., 1974):

$$P_d = 2(k - 1) \rho q_v \times 10^{-4} \quad (\text{in kg/cm}^2), \quad (\text{C-1})$$

$$D_d = \sqrt{2(k^2 - 1) q_v} \quad (\text{in m/sec}), \quad (\text{C-2})$$

$$U_d = \frac{D_d}{k + 1} \quad (\text{in m/sec}), \quad (\text{C-3})$$

where P_d is the pressure due to VCE explosion; ρ_0 is the density of the explosion mixture in $\text{kg} \cdot \text{sec}^2 / \text{m}^4$; $k = C_p / C_v$ is the heat capacity ratio; C_p is the heat capacity of the gas-air mixture; C_v is the heat capacity of the explosion products; and q_v is the specific heat of the explosion.

The heat capacity ratio, $k = 1.25$, has been estimated (Andreev and Beliaev, 1960; Baum et al., 1959) for most inflammable materials (methane, ethane, butane, pentane). In estimating the VCE detonation process, the radius of the hemisphere of vapor cloud zone (zone I) is calculated from the following equation

$$r_0 = 0.78 \sqrt[3]{\omega G_g} \quad (\text{C-4})$$

where r_0 is the radius of the volume of the VCE; ω is the volume of the VCE, generated from 1 kg of product, in m^3 / kg (for hydrocarbon gases the magnitudes of $\omega = 13.25 - 16.2 \text{ m}^3 / \text{kg}$); and G_g is the mass involved in the VCE.

In zone I, the almost-permanent overpressure, ΔP_d , ($r \leq r_0$) is produced based on the properties of the explosive material. For most types of hydrocarbon gases, the overpressure is equal to $\Delta P_d = 15 - 18 \text{ kg/cm}^2$ (Bodansky et al., 1974).

In the case of spherical explosion (without barrier in the surrounding environment) of the VCE, the limit radius of the explosion (Zone II) is be given by:

$$r_1 \approx 1.7r_0 \quad (\text{C-5})$$

The overpressure, ΔP_d , in zone II ($r_0 < r \leq r_1$) is determined by:

$$\Delta P_d \approx 13 \left(\frac{r_0}{r} \right)^3 + 0.5 \text{ , (in kg/cm}^2\text{)} \quad (\text{C-6})$$

The effective time duration of the explosion action for zone I ~ II is given by:

$$\theta = 0.47 \times 10^{-3} r_0 \left(1 + \frac{0.4 r}{r_0} \right) \text{ (in sec), for } (r \leq r_0) \quad (\text{C-7})$$

and

$$\theta = 1.82 \times 10^{-4} r_0 \sqrt{\left(\frac{r}{r_0} \right)^5} \text{ (in sec), for } (r_0 < r \leq r_1) \quad (\text{C-8})$$

When the distance is more than r_1 (Zone III), the shock wave "breaks away" from the explosion, and the pressure becomes independent of the overpressure in the center zone of VCE.

The overpressure, ΔP_d , in zone III ($r_2 \geq r_1$) is determined by:

$$\Delta P_d \approx \frac{7}{3(\sqrt{1+29.8R_2^3} - 1)} \text{ , (in kg/cm}^2\text{)} \quad (\text{C-9})$$

Where R_2 is the equation for dimensionless shock wave radius, given by:

$$R_2 = 0.24 r_2 / r_0 \quad (r_2 \geq r_1) \quad (\text{C-10})$$

The effective time duration of the explosion action for zone III is:

$$\theta = \frac{2.5 r_2 \times 10^{-4}}{\Delta P_d R_2^2} \text{ (in sec) for } (r_2 \geq r_1) \quad (\text{C-11})$$

Note that r_2 is not a limit radius as r_0 and r_1 are.

The equations in Section 4.1 were used for estimating the possible effect of VCE to WTC, as discussed in Chapter 4.

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