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Investigation of Fire Impact on Structural Steel through Case Studies

A Thesis Report

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by

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Abstract

Death of firefighters due to structural collapse has been on the rise for the past few years, and has not gone unnoticed by the research and firefighting branches of the industry. However, the modes for improving this situation by both are very different. While firefighters depend on experience for detection, research organizations have invested in developing new technology to detect signs of structural collapse. Thus far neither effort has led to any improvement in the current circumstances. In order to bridge this gap, members of the fire-safety community need to more thoroughly understand the reasons for structural collapse due to fire. Through research and analysis, a case study manual analyzing structural steel failures due to fire was developed. This manual contains analysis of the actual mode of failure for the cases chosen, as well as analysis of alternative situations for each case that may have led to different outcomes. The goal of this manual is to aid in the teaching and practice of structural steel collapse due to fire as a supplement to current knowledge.

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1 Introduction

1.1 Project Development

This project was developed as a piece of a larger research project, "Structural Collapse Assessment and Visual Cues for the Fire Service: Framework and Prototype Development" to be started in June of 2006. The proposal for the overall project recognizes the void in communication between the engineering community and fire services and will attempt to diminish this by creating a first-generation, computer – based tool that supports gathering and organizing project – specific information on structural anatomy and different forms of building construction and relates this information to descriptions of structural performance and possible collapse scenarios. In order to accomplish its objectives, researchers for the proposal discussed above will focus on pre-incident planning.

From the Structural Collapse Assessment proposal came the concept for the research and analysis performed for the project discussed herein. While taking into account the trends in firefighter fatalities over the past thirty years, and the lack of communication between the engineering research community and the fire services, it became apparent that there is a fundamental building block missing in the education of both professions. Fire programs, schools and academies nationwide teach up and coming firefighters about the fundamentals of building construction and fire behavior in buildings, but they do not address the fundamentals of how such buildings collapse. Several reasons for this, as Dunn (1988) states in the first chapter of his book, are the inability of the fire service personnel to objectively analyze a collapse after the death of one of their own, the lack of a standard definition for collapse in the fire service, and the lack of recorded data pertaining to structural collapse for use by the fire service. Similarly, academia teaches engineering students how to design structures at ambient temperatures but does not teach how to design for fire conditions, which is in part due to the fact that structural engineers are not currently responsible for the structural fire safety of buildings.

One way to remedy this is for structures to be designed for fire conditions, which is outlined in *2005 AISC Specification for Structural Steel Buildings, Appendix 4: Structural Design for Fire Condition.* However, this is not the method currently taught or practiced by the engineering community. In order to supplement current curriculum/knowledge, case studies can be a useful tool. There have been many articles written on the effectiveness of using case studies as an educational tool. For instance, one paper (Rens, 2000) states that "case studies can be a useful teaching aid that promotes higher levels of cognitive thinking and learning through examples."

1.2 Project Description

The project objective was to develop a case study manual that investigates structural steel systems that have collapsed during fire conditions. In addition, alternative situations that could have caused different outcomes will lend themselves to supplement a small piece of the structural design education in the field of structural collapse and how structural members behave under collapse conditions. The goal of this manual is to introduce how structural steel members can fail by way of fire and to demonstrate application of the tools and methods available for these performance analyses. While the cases are specific to certain structures, the means of analyses used can be applied generally. Investigation of steel in this manner will serve to increase awareness of its material properties and behavior under fire conditions and contribute to the teaching and practice of performance based structural design for fire conditions and to stimulate discussion of the subject area and promote interest in studying the subject further.

2 Background

In this chapter, the topic of firefighter safety and education is researched to put the importance of structural collapse due to fire into perspective. The trends in firefighter fatalities show an overall decrease, but an increase in the number of deaths due to structural collapse. Training for structural collapse should come through education and experience; however this does not seem to be the case when reviewing the educational material taught to firefighters. In order to remedy this in the fire service, teaching material about the properties and failure modes of materials should be supplemented into the education. To improve this from a structural engineer's standpoint, designing structures for fire is necessary, and seems to be the rising trend, as seen in the *2005 AISC Specification for Structural Steel Buildings, Appendix 4: Structural Design for Fire Condition, Appendix 4: Structural Design for Fire Conditions.* In order to supplement current engineering curricula to incorporate this type of design, case studies can be utilized to teach individuals of different failure modes associated with different cases and the analyses performed to get those results. Case studies can also be an effective way for teachers to promote individual learning outside the class room. To conclude this chapter, a section has been included that describes state-of-the-are technology being developed by NIST, FEMA, and the USFA to predict structural collapse.

2.1 Firefighter Safety and Structural Collapse

"Between the years 1979 and 2002 there were over 180 firefighter fatalities due to structural collapse," (Brassel, 2003). In 2003, the National Institute of Standards and Technology (NIST) prepared a report investigating the "Trends in Firefighter Fatalities

Due to Structural Collapse" in collaboration with the United States Fire Administration (USFA). This report analyzed data from several different studies performed between 1979 and 2002. The results relate different parameters taken from the previous studies to determine any relationships between, for example, cause of death and a specific fire fighting activity or event. The study found that while firefighter fatalities due to collapse have decreased since 1979, the number of deaths "caused by being caught or trapped in the structure has increased," (Brassel, 2003). It must be noted that the classification for "caught or trapped" includes cases that were once classified as "fell or jumped" in the USFA "data collected for the years 1994-2002," (Brassel, 2003). Figure 1 below shows the percentage of fatalities from the years 1979-2002 by being "caught or trapped" or "struck by/contact with object."

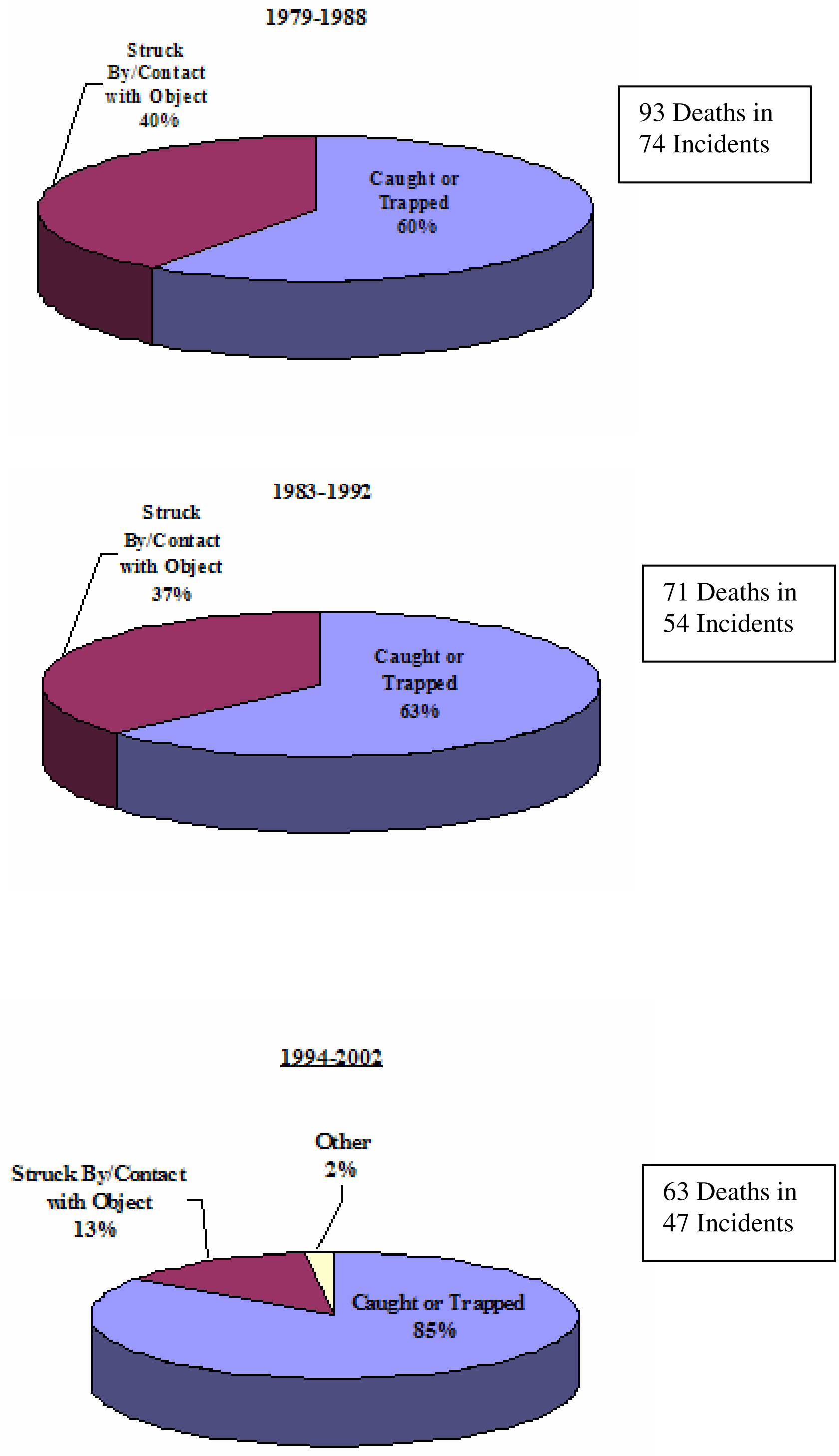

Figure 1 – Fatalities by Cause of Death (Brassel, 2003)

One major result that came from this study that directly affects this research project is the fact that the number of fatalities did not decrease drastically with fire fighter experience. This can be seen in Figures 2 and 3 below, extracted from the NIST report "Trends in Firefighter Fatalities Due to Structural Collapse, 1979-2002," which compare the number of firefighter fatalities with experience and rank.

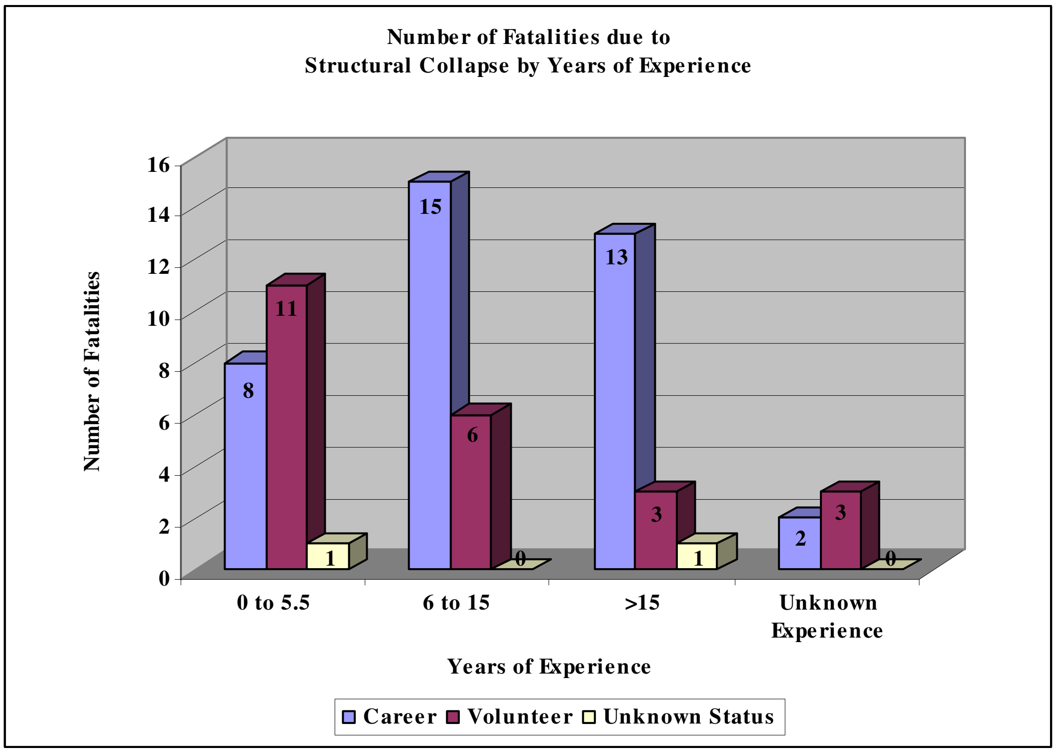

Figure 2 – Fatalities by Years of Experience (Brassel, 2003)

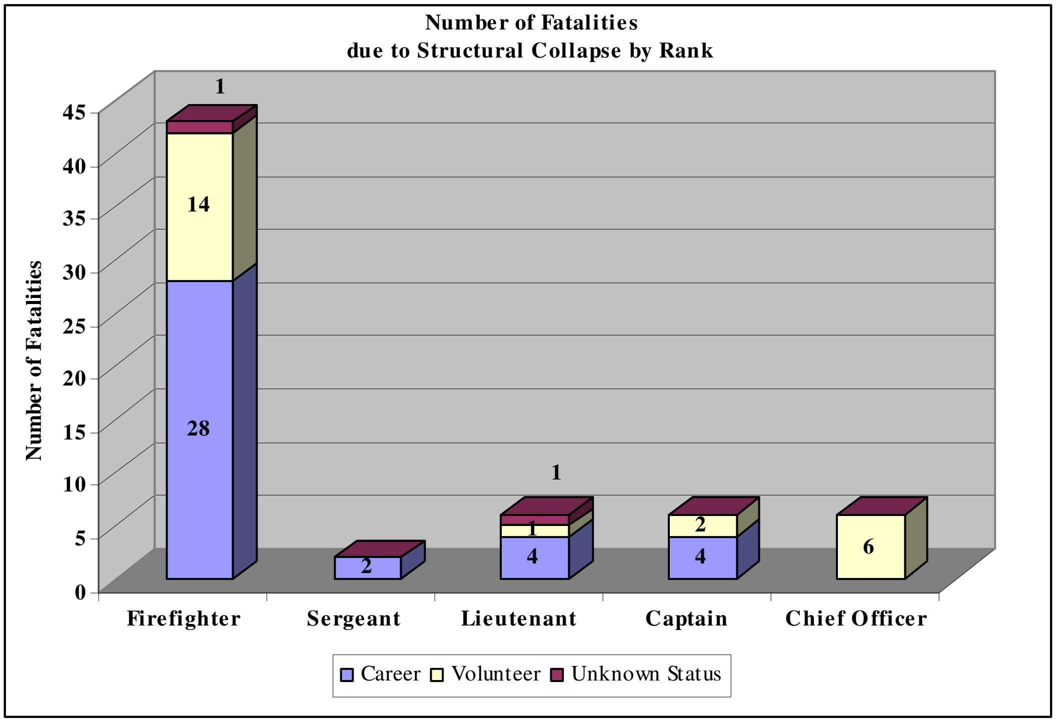

Figure 3 - Fatalities by Rank (Brassel, 2003)

This result is very important because when it comes to firefighters' knowledge about structural collapse many say they rely on experience. One WPI – Interactive Qualifying Project team that interviewed local fire officials reported that the officers continue their education through classes and certification programs, but much of their knowledge in assessing a building's structural performance came from experience (LaMalva, 2005).

2.2 Guidelines for Fighting Structural Fires

Several warnings and guidelines have been issued to establish procedures for structural firefighting that are intended to reduce the risk of death due to structural collapse.

After a similar report to the one listed above was published by the NFPA in August of 1999, the National Institute for Occupational Safety and Health issued a warning to fire departments across the country of the dangers of structural collapse. This warning contained eleven points that should be met by the fire services in order to "minimize the risk of injury and death to firefighters during structural firefighting," (NIOSH, 1999). These steps ranged from pre-incident planning to keeping lines of communication open and establishing escape routes.

The Occupational Safety & Health Administration also publishes Emergency Preparedness Guides to warn about structural collapse. This guide is intended to address not only structural collapse due to fire, but also due to natural disasters and terrorist attacks. The guide defines interior and exterior structural collapse due to natural causes, such as hurricanes, tornadoes etc. and also from explosions. It also states that "emergency responders and rescue workers" are the people who generally enter a building after a collapse and outlines some of their duties, which may include, "assisting survivors, extinguishing fires," etc, (OSHA, 2005). The guide also states "site management will eventually be under an Incident Command System. Local responders and rescuers will obviously respond first with the State requesting Federal Emergency Management Agency (FEMA) assistance if warranted," (OSHA, 2005). The guide then lists the duties of the Incident Commander and Urban Search and Rescue personnel. It concludes by listing a number of safety hazards that could be encountered by those entering the collapsed structure, such as, flooding, exposed wiring, gas leaks, structural instability, etc…

One concept that has been developed and used by fire departments across the country is the LCES concept. This stands for Lookouts, Communications, Escape Routes, and Safety Zones. This concept was developed by "highly experienced firefighters who, in studying wildland fires that resulted in firefighter deaths, recognized certain patterns that contributed to deaths," (Collins, 2002). This concept has spread to not only combating wildland fires, but also structural fires, and other operations, such as search and rescue. The concept is simple: set lookouts at strategic points to evaluate hazards, set up clear communications for workers, establish escape routes, and identify safety zones. These four steps may help warn workers of hazardous conditions, enlighten the officers in charge of workers' locations, and enable workers in danger to get to escape routes and then to safety. While the LCES system seems simple, without having these four steps established, many firefighters and rescue workers have died and will continue to die in firefighting operations.

The three guidelines listed above are just a few of the many ways in which fire departments are advised to fight fires. Most fire departments have their own system for protocols that may adapt or reflect these guidelines. Protocols will help to reduce the risk due to structural collapse, but an increased knowledge of how structures behave will serve to reduce that risk more.

2.3 Firefighter Education

The IQP, *Understanding and Enhancement of Structural Engineering Principles Incorporated in Fire Department Databases and Education (2005)* there are three books that are most commonly used by the fire service to teach firefighters about building construction and fire safety. These books are <u>Building Construction for the Fire Service</u> by Francis Brannigan, <u>Collapse of Burning Buildings, A Guide to Fireground Safety</u> by Vincent Dunn, and the <u>Firefighter's Handbook: Essentials of Firefighting and Emergency Response</u> by the National Fire Academy Alumni Association.

For the research purposes of this project, these books were analyzed to determine the amount and detail of information regarding structural steel properties and behavior in terms of collapse.

2.3.1 Building Construction for the Fire Service, 2nd Edition (Brannigan)

This book begins with the chapter on Principles of Construction. This chapter details the importance of studying building construction, stating "the most important reason for knowing building terms is safety," (Brannigan, 1982) and then gives a discussion of the different load cases found in buildings, such as dead, live, concentrated, etc… It then proceeds into a basic description of the characteristics of different building materials and structural members.

Skipping ahead to chapter seven brings the reader to the discussion of steel construction. The chapter begins with a description of the basic characteristics and properties of steel as a metal, such as the coefficient of expansion, the yield point, and the ability to conduct heat. It then leads to several definitions for structural steel members, such as I-beams, channels, plates, etc…

The chapter on steel construction also provides an in-depth discussion of the importance of protecting steel from fire, which can be done by steel fireproofing, equipping a building with sprinklers, or designing the structure so the steel will be out of the range of heat produced by the fire. The standards for protecting a building against fire are set by local and national building codes. The book provides several case studies to analyze what happened in buildings that did not have the appropriate fire suppression systems, such as the McCormick Place Fire in Chicago, Ill. It also provides many pictures to illustrate the points being discussed, however these points could be better illustrated with arrows.

Chapter eleven of this book describes high-rise construction. The book describes the different building construction methods used in high-rise structures from the years 1870 to the present, and it illustrates the deficiencies with each type of building construction. This is an important chapter to the research of this project since many of the high-rise collapses in this country have been constructed of steel.

While this book details and illustrates many properties of building construction for steel construction, it does not go into the discussion of how the structural members and systems may react physically and mechanically when exposed to different fire conditions.

2.3.2 Collapse of Burning Buildings, A Guide to Fireground Safety (Dunn)

This book begins with an introduction of building collapse and an explanation for the lack of recorded data on the subject. The author states that there are several reasons for the lack of information about structural collapse, which include "research into the subject offers small benefit to anyone except firefighters," and firefighters are usually incapable of objectively analyzing the collapse (Dunn, 1988). Other reasons are the lack of a formal definition of structural collapse within the fire service, and the lack of recorded data by fire departments across the country.

This book, like Brannigan's, devotes a chapter to building construction terms and provides effective pictures to illustrate the building concepts. The text also details the hazards associated with each of the five general types of building construction. Unlike Brannigan's book, this text does not devote a chapter to steel construction. However, chapter ten does discuss the concept of lightweight steel roof collapse and the hazards associated with it.

Each chapter of Dunn's book gives detailed descriptions of situations that firefighters have encountered while fighting fires in the different forms of building construction. Each example gives studying firefighters some insight into a hazard associated with that type of construction. The book also offers good illustrations of the construction types, and a "lessons learned" section at the end of each chapter.

2.3.3 Firefighter's Handbook: Essential of Firefighting and Emergency Response, 2nd Edition (Thomson Delmar Learning)

While Dunn's book focuses on collapse in different structures, and Brannigan's book focuses on building construction, this book covers all of the essentials that may be taught to firefighting students, this is a true textbook for the fire service. Like Dunn's book however there is no chapter dedicated to steel as a building material, just chapter thirteen dedicated to building construction.

This chapter begins with one fire chief's account of a structural collapse and then goes into the factual knowledge about building construction. It begins by defining loads and structural elements. The text then leads into a brief discussion of the four main building materials, wood, steel, concrete, and masonry and states how the materials react to fire, which are summarized in a table, as can be seen in Figure 4 below.

Performance of Common Building Materials under Stress and Fire				
MATERIAL	COMPRESSION	TENSION	SHEAR	FIRE EXPOSURE
Brick	Good	Poor	Poor	Fractures, spalls, crumbles
Masonry block	Good	Poor	Poor	Fractures, spalls
Concrete	Good	Poor	Poor	Spalls
Reinforced concrete	Good	Fair	Fair	Spalls
Stone	Good	Poor	Fair	Fractures, spalls
Wood	Good w/grain; poor across grain	Marginal	Poor	Burns, loss of material
Structural steel	Good	Good	Good	Softens, bends, loses strength
Cast iron*	Good	Poor	Poor	Fractures

*Some cast iron may be ornamental in nature and not part of the structure or load bearing.

Figure 4 - Performance of Common Building Materials (Thomson, 2004)

It also discusses new building materials, such as composites and discusses some of the problems related to emerging materials. The chapter then progresses into the five types of building construction, fire-resistive (I), noncombustible (II), ordinary (III), heavy timber (IV), and wood frame (V), and a discussion of newer construction types, such as lightweight steel. This section also discusses the types of occupancies associated with the five types of construction and lists hazards associated with these occupancies.

Typical Hazards Associated with Occupancies		
OCCUPANCY	TYPE OF CONSTRUCTION	HAZARDS
Residential	Type V, most common	Fire loading, truss construction, owner alterations, rapid fire extension in void spaces
Commercial	Type III, most common	Fire loading, truss construction, rapid fire extension in void spaces, unknown occupancy change
Educational	Type II, most common	Unprotected structural steel, collapse, high fire load in some areas
Business	Types II and III, most common	Unknown change in occupancy, high fire load, difficult to ventilate
Industrial	Types I and II, most common	Hazardous materials, difficult to ventilate

Figure 5 - Occupancy Hazards (Thomson, 2004)

The chapter then addresses the issue of structural collapse. While it does not address every issue, it focuses on trusses, roofs and ceilings, stairs, parapet walls, and highlights the hazards of structural collapse for a building under construction. This section also provides a list of "collapse signs." The text concludes with a "Lessons Learned" section and glossary of key terms and their definitions used in the chapter.

This book provides firefighter's with a wealth of knowledge and provides real life accounts and examples, color pictures, tables and "streetsmart tips" to help illustrate the key points. Additionally, this book goes farther than the others in describing material performance and structural collapse. It also drives home the point that firefighters need to be actively inspecting buildings in their jurisdictions to know the building construction in their areas. While this book provides firefighters with a lot of knowledge on building construction there is still much it does not cover. As can be seen from the books discussed above, a whole book could be dedicated to the subject.

While all three books are good resources for educating firefighters about building construction and the associated safety concerns, they provide minimal insight into the actual behaviors of the structural members and systems used in the types of construction. A supplemental resource to provide this information is needed to offer a better understanding of the behavior of the elements of structures. These books also provide the engineering community an insight into the education of firefighters and the importance of studying as well as designing for structural collapse.

2.4 Education through Case Studies

For both the fire protection service and the engineering community, case studies provide a useful tool as a supplement for education. "One common belief is that failure case studies can be a useful teaching aid that promotes higher levels of cognitive thinking and learning through examples," (Rens, 2000).

One paper, *Forensics and Case Studies in Civil Engineering Education: State of the Art* (Delatte, 2002) illustrates the various ways in which case studies can be utilized for educational purposes; case studies can be used as a course, as is taught at Worcester Polytechnic Institute, MA, and Massachusetts Institute of Technology, MA. They can also be used to teach a forensic engineering course as is done at the University of Colorado, Denver, University of Texas, and Mississippi State University. The material could also be integrated into existing classes to fortify course material being taught, or to aid with the completion of capstone design courses.

In 1982, the American Society of Civil Engineers (ASCE) Technical Council on Forensic Engineering (TCFE) was formed. The goals of this council were to (Delatte, 2002):

- "Develop practices and procedures to reduce failures;
- Disseminate information of failures and there causes, providing guidelines for conducting failure investigations;
- Encourage research and education in forensic engineering; and
- Encourage ethical conduct in forensic engineering practice."

The council performed two surveys of civil engineering departments at accredited universities across the country in 1989 and 1998 to determine the use of failure analysis in education. From the questionnaires came the main point that a "lack of instructional materials" (Rens, 2000) was the main reason for the lack of inclusion into the curricula.

In order to solve this problem, the University of Colorado, Denver developed a website to provide professors and students access to failure data. The site was constructed using information from books, technical reports, journals, television and other credible sources. While the site is a pilot and is still under construction, it invites others to provide cases to be added to the site.

The use of case studies in education is a useful tool to illustrate the effects of design gone wrong and the implications of those failures. “It has become clear how little students generally know about key disasters that are common knowledge to their elders and that have had a profound effect on the profession,” (Jennings, 2000). Both at the undergraduate and graduate levels, case studies should be implemented into the curriculum to supplement and expand students’ knowledge.

2.5 Structural Design for Fire Conditions

The structural design of buildings involves analysis of the members involved for loads, such as gravity, live, wind, and earthquake loads. This design does not often include designing these members for fire conditions. This, in part, is due to the fact that structural engineers are not currently responsible for the integrity of the structure when exposed to fire. However, a shift in responsibility may be on the horizon.

In the *2005 AISC Specification for Structural Steel Buildings, Appendix 4: Structural Design for Fire Condition, Appendix 4: Structural Design for Fire Conditions* specifies means by which members should be designed to account for the effects of fire. The performance objective is stated as “structural components, members and building frame systems shall be designed to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy,” (AISC, 2005). The appendix discusses a number of different fires and material properties. It then goes into the methods for analysis, which include simple analysis (lumped heat capacity analysis), advanced analysis (using computer modeling), and qualification testing. The "lumped heat capacity" analysis method was utilized in this project to determine the temperatures of the steel members analyzed in the cases chosen with various thickness and application of three different design fires.

The importance of case studies in education as detailed above, and the new design requirements emerging require students and designers alike to gain a better understanding of how structures react when exposed to fire. Through this project, four cases were determined and analyzed to provide data in this area. While the cases are specific to certain structures, the means of analysis used can be applied generally. The intended goal of the case study manual developed is to contribute to the teaching and practice of performance-based structural design for fire conditions and to invoke interest in this field.

2.6 Research and Technology

Over the past few years, researchers have turned their focus to the matter of structural collapse. The National Institute of Science and Technology has developed many studies to advance the industry's knowledge in several areas pertaining to structural collapse.

After the collapse of the World Trade Center towers on September 11, 2001, NIST investigated how the towers collapsed. For this report and for others, such as the Rhode Island Station Nightclub (also investigated by NIST), analysis using a Fire Dynamic Simulator and Smokeview modeling were used. To establish these models researchers considered "fire growth and spread, the impact or potential impact of fire safety systems or changes in egress arrangements, and the conditions building occupants and firefighters encountered," (Dittmar, 2005). The information for the factors considered was obtained through "photographs and videos, recovered steel, eyewitness accounts and emergency communication records," (Newman, 2005). These software packages provide three-dimensional modeling capabilities and were utilized in studies done on the behavior of occupants escaping buildings, and on the techniques of structural ventilation.

NIST has also developed a research project to focus on structural redundancy and the mitigation of progressive structural collapse. In collaboration with the United States Fire Administration (USFA), NIST has performed a series of full-scale fire tests in four, one – story wood frame structures. These tests were conducted in order to determine "the feasibility of predicting structural collapse" (Stroup, 2004).

Another research project was also done using full-scale fire tests of wooden and lightweight steel building. The goal was to test a device developed to measure the vibrations of burning structures. Researchers were aiming to discover a way to predict structural collapse based on the vibrations of the structure using accelerometers. While the device is still in the testing phases, it may become a vital tool for firefighters in the future (Duron, 2003).

NIST, as well as FEMA, and the USFA have been conducting many studies to advance the industry's knowledge of structural collapse and steps that may be taken to reduce this risk to those involved. While collapse detecting devices are on the horizon, 3-D modeling software has proved very effective in the analysis of fire conditions.

3 Literature Review

Steel and concrete are the two main materials used for building high-rise structures. Structural steel is generally used to build larger buildings due to the cost of material and labor, which is why most of the cases involving steel collapse are multi-story structures. One issue that has been discussed since the 1970's is the need for automatic sprinkler systems to be incorporated as a feature of high-rise buildings. Only one of the cases investigated through this project was equipped with an automatic sprinkler system, and in that case it failed to operate under the exposed fire conditions.

3.1 McCormick Place

McCormick Place was a large exhibition hall that stood on the shore of Lake Michigan in Chicago, Ill. It was constructed in 1960 of reinforced concrete and structural steel and consisted of 3 levels of exhibition space, a large theater, restaurants, and a variety of other rooms and supporting spaces. Large steel trusses supported the roof of the structure; they spanned 210 feet column to column and cantilevered 80 feet on either side, as can be seen below in Figure 6.

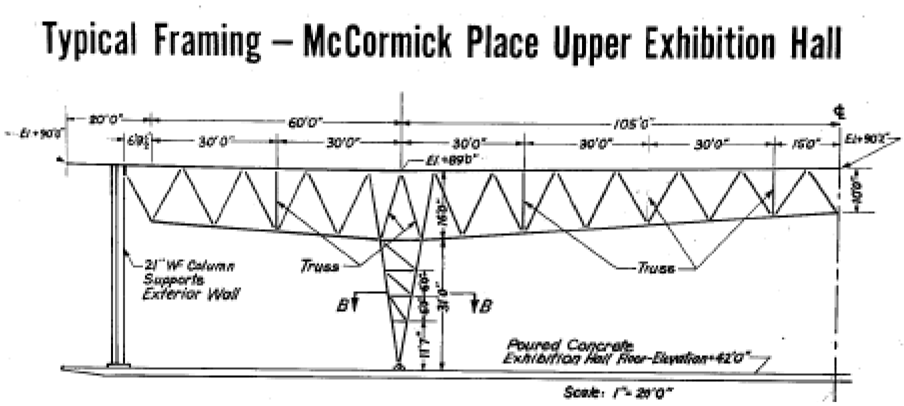

Figure 6 - McCormick Place Roof Truss (Jensen, 1967).

At the time of the fire, McCormick Place was hosting the National Housewares Manufacturers Association Show, which occupied two levels of the hall. The booths for this show were constructed of plywood, fabrics, and other combustible materials that produced a large fire load. The fire began at around 2 a.m. on January 16, 1967 as an electrical fire. It quickly consumed the entire 3rd floor and spread down to the 2nd floor by way of melted expansion joints. The exhibition portion of the hall was not protected with automatic sprinklers and the exposed structural steel was not protected from fire exposure. Lack of fire protection caused excessive heating of the truss members, which then led to the collapse of the roof (Jensen, 1967).

3.2 World Trade Center 5

World Trade Center 5 was a nine-story office building that was completed around 1970. Fire in this building broke out due to debris that impacted the building from the collapse of the World Trade Center towers on September 11, 2001. The subsequent fires caused shear connectors of the column tree assemblies to fail initiating collapse from the eighth floor through the fifth floor. The roof and ninth floor (which contained no column trees) as well as the fourth floor and floors below experienced no collapse. The column tree assemblies and conventional framing systems can be seen in Figures 7 and 8 below.

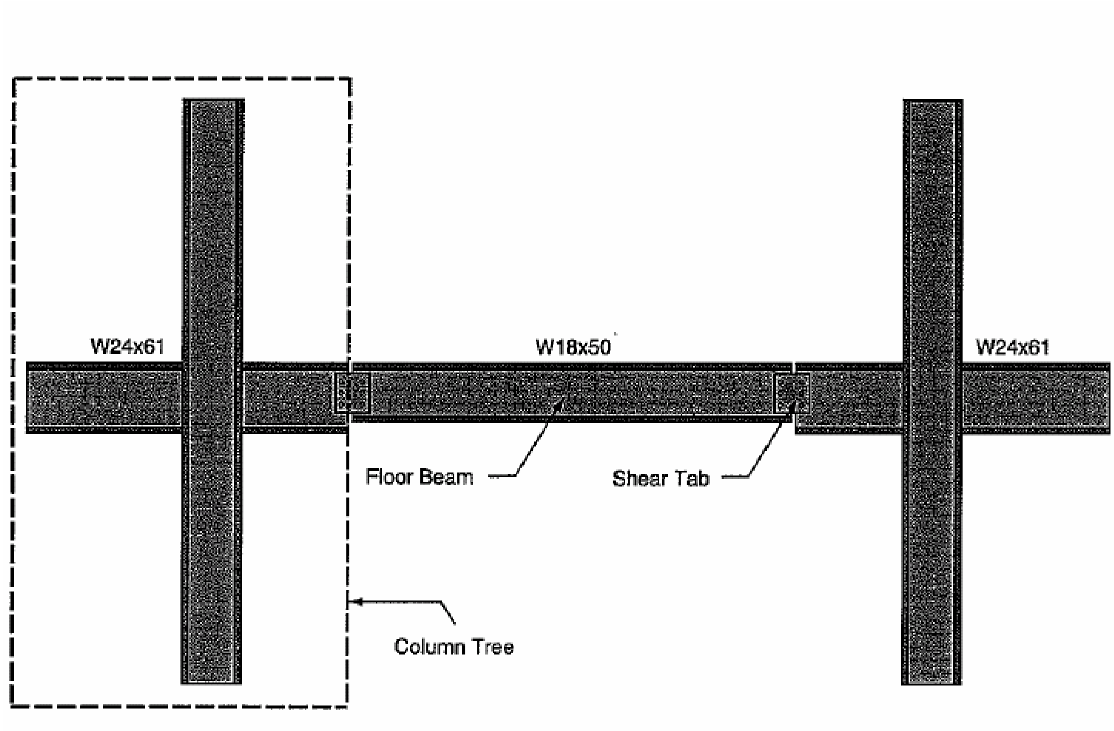

Figure 7 - Typical Column Tree System (Barnett, 2002)

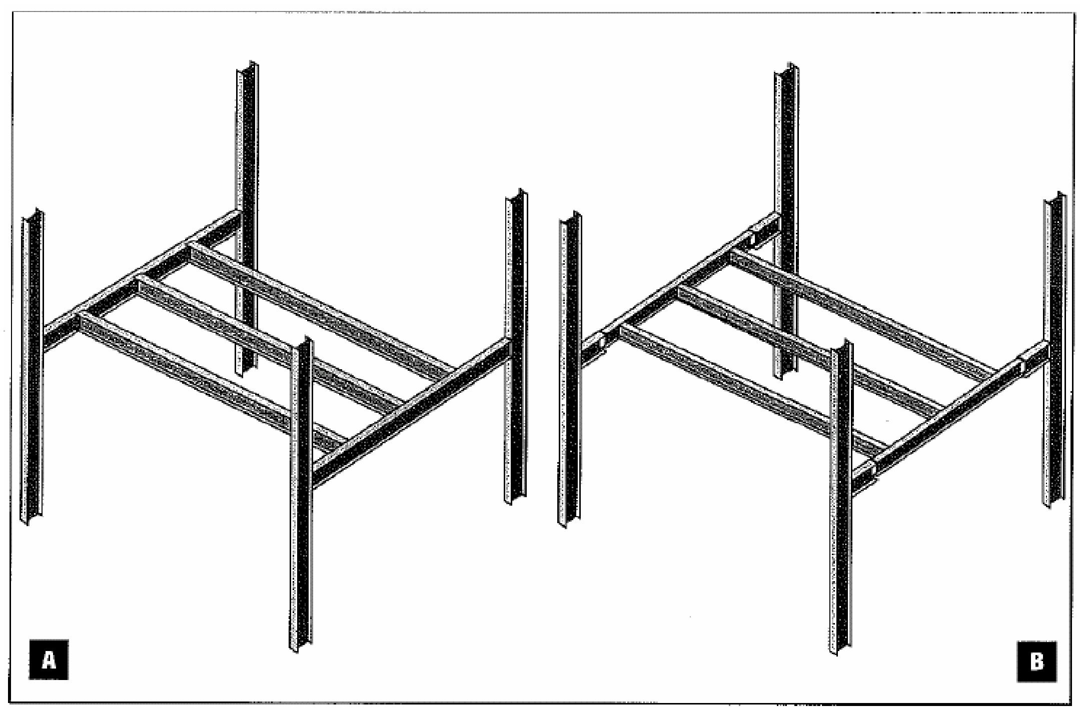

Figure 4-3 Typical interior bay framing in WTC 5. (A) Floor 9 and roof level. (B) Floors 4, 5, 6, 7, and 8.

Figure 8 - Interior Bay Framing of WTC 5 (Barnett, 2002)

Due to the havoc of the day, the fire burned uncontrolled for several hours. For reasons unknown, the automatic sprinkler system did not activate to aid in the suppression of the fire (Barnett, 2002).

3.3 Alexis Nihon Plaza

The ten-story office building (referred to as the 15-story office building in the official report) that was part of the Alexis Nihon Plaza was built atop a five-story shopping/parking facility sometime after 1950. On the evening of October 26, 1986 at around 5 p.m., a fire broke out on the 10th floor and spread through stairwell B (see Figure 9) up to the 16th floor (there was no 13th floor). Although the source of the fire was undetermined, it was believed to have started in a communications cabinet next to stairwell B.

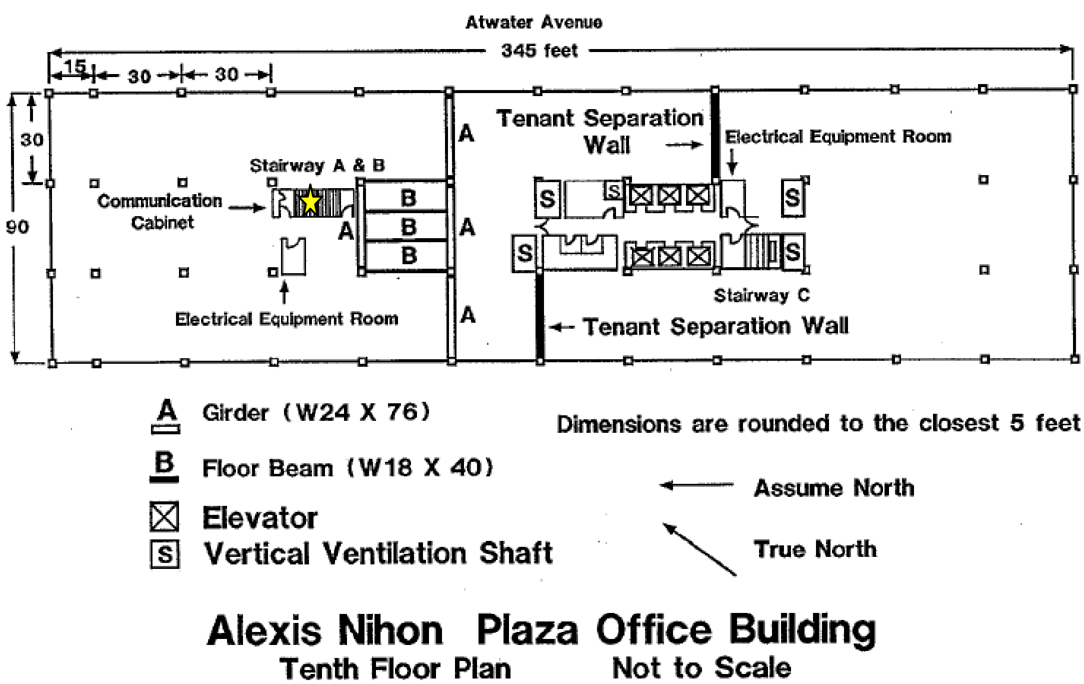

Figure 9 - Alexis Nihon Tenth Floor Layout

“Most of the areas in the office tower were used for general clerical work. However, these areas had considerable amounts of storage in addition to the materials stored in the space designated for that use,” (Isner, 1986). The additional storage material kept could have attributed to the rapid spread and intensity of the fire. The fire burned for “13 hours and 24 minutes after the first alarm” (Isner, 1986). Fire fighting efforts were hindered by the lack of water pressure in the standpipes in and around the building.

At approximately 10:30 p.m., a 30 ft by 40 ft. section of the 11th floor, near where the fire was believed to have originated, collapsed onto the 10th floor (Isner, 1986). This was the only section of the building that suffered collapse, and upon investigation, the collapsed members did not exhibit the properties commonly found of members exposed to high temperatures, “such as bending, elongation, or twisting,” (Isner, 1986).

3.4 One New York Plaza

One New York Plaza is a 50-story office building that was completed in early 1970. On August 5, 1970 a fire broke out on the 33rd floor, which was believed to have started in a telephone equipment room. The fire then spread to the 34th floor by means of air-conditioning duct openings. The fire thrived for five hours on the polyurethane furnishings before being controlled by local authorities. The high temperatures reached during the fire caused failure of connections and distortion of the beams and girders supporting the 34th floor. Since the building had just been recently completed, it had not yet been fully occupied.

The New York Board of Fire Underwriters made several recommendations as a result of this fire which were intended for all buildings similar to One New York Plaza, high-rise office structures. These recommendations included banning the use of “flammable foamed cushioning,” reducing the fire load allowed in fire-resistive buildings, protecting steel members with materials that cannot be readily removed, and providing fire stops between floors (Powers, 1970). Many of the recommendations provided through this report were incorporated into later building codes.

The four cases listed above were all constructed of structural steel in some manner. Through the investigative reports of these cases, details of the structural systems were obtained. Analysis on these systems was then done to determine the effects of fire on structural steel and will be discussed in the Sections that follow.

4 Methodology

To complete this research project, cases had to be chosen to analyze the effect of fire on structural steel. Through this section one can see how each case was determined and the analysis done on the cases to produce the results that appear in Section 5.

4.1 Case Study Research

In order to determine appropriate case studies to be analyzed for this project, parameters had to be set. These parameters were at first general and then became more selective as the research process continued. To begin the search, the requirements were cases constructed of a structural steel frame and had experienced a full or partial collapse due to fire exposure.

To complete this search, Internet search engines, such as, Google Scholar, Google, and Yahoo were used. Other resources included books, journals (*Fire Journal*, for example), professional websites (USFA, NFPA, etc…), online databases (LexisNexis, Firedoc, etc…) and information from individuals. This broad search resulted in several papers, studies and reports that noted structural collapses dating back to the 1960's.

One study done by NIST, "Analysis of Needs and Existing Capabilities for Full-Scale Fire Resistance Testing," which was completed in December of 2002 by Beitel and Iwankiw, presented an in-depth investigation into the structural collapses of multi-story buildings (4 or more stories) in North America and around the world from the 1950's to the present. The report collected information from news sources, online databases and professional organizations. The information sought was the building name, location, type of construction, building height, date and time of collapse, and the extent of collapse, which were then tabulated. The NIST report also provides information on major fires in multi-story structures that did not experience collapse but suffered major structural damage. The information from this report was important in identifying several cases to further research.

Other cases identified for further research Francis Brannigan's book. In *Building Construction for the Fire Service*, *2nd Edition* chapters 7, Steel Construction and Chapter 11, High Rise Construction provided several cases for investigation. Two of these cases were later chosen as cases to be analyzed through this project.

4.2 Determination of Cases

After a number of cases had been initially investigated, further research of each of the cases was then completed. The initial cases investigated are listed below in Table 1.

Case	Location	Date
McCormick Place	Chicago, Ill	1/16/1967
7 World Trade Plaza	New York, NY	9/11/2001
5 World Trade Plaza	New York, NY	9/11/2001
Alexis Nihon Plaza	Montreal, CAN	10/26/1986
Couer de Royale Condominium	Couer de Royale, MO	8/25/1994
Effingham Plaza Nursing Home	Portsmouth, VA	4/6/1998
Nelson Morris Company	Chicago, Ill	1910
General Motors Corp.	Livonis, MI	8/1953
Marine Corps Depot of Supp.	Norfolk, VA	1946

Table 1 - Initial Cases Identified for Investigation

It proved difficult to find information on many of the cases originally identified due to lack of recorded data. From this research, seven cases were identified which had detailed technical reports about the fire and collapse of the structures. These included the World Trade Center 1, 2, 5, and 7; Alexis Nihon Plaza; One New York Plaza; and McCormick Place.

The controversy associated with the technical reports about the World Trade Center towers, buildings 1 and 2, and building 7 ruled out these cases for further investigation, which left the four others previously mentioned. The structural information and knowledge about the fires provided in those technical reports are the basis for the analysis performed here in. The issues investigated for each case are listed below in Table 2.

Case	Element Studied
World Trade Center 5	Floor Framing- Original Alternative Framing Systems Shear Tab Connections
Alexis Nihon Plaza	Welded Angle Connections Bolted Angle Connections
McCormick Place	Structural Steel Trusses
One New York Plaza	"Lessons Learned"

Table 2 - Case Studies & Elements Investigated

4.2.1 McCormick Place – Overview

There were 18 structural steel trusses that supported the roof of this building. These trusses, as discussed in Section 3.1, spanned 210 feet column to column and cantilevered 80 feet on either side. The trusses were 16' deep above the rigid frame support, 10 feet deep at the center, and 5'-3-1/2" deep at the cantilevered end, please see Figure 6 in Section 3.1. The truss and rigid frames that supported them were constructed of W14 wide flange members and were stabilized by smaller trusses running perpendicularly similar to bridging. The column portion of these frames were protected from fire with a sprayed-applied fiber and encased "with metal lath and Gypsum vermiculite plaster," (Jensen, 1967) up to a height of 20 feet, but the trusses did not have any fire protection applied to them.

Once the fire broke out, it spread rapidly through the exhibition hall. With the large fire load present and an inability to obtain water from the exterior fire hydrants, fire-fighting efforts proved futile. The unprotected steel of the trusses could not withstand the excessive temperatures reached by the fire and resulted in a roof collapse approximately an hour after the fire began. It was stated that "the roof trusses started to buckle in the center, pulling the roof loose from the columns at the walls," (Juillerat, 1967).

This case was studied first and became the base case for the analyses performed for the others. The reason for this was that through study it clearly demonstrated the effectiveness of insulation. The calculations for this analysis were performed using the equations presented in Section 4.3. While this was the first step in the analysis of the other cases, it was less important because in those cases the steel had been protected with insulation prior to the fires.

4.2.2 World Trade Center 5 – Overview

World Trade Center 5 was a nine-story office building constructed of a structural steel frame and composite floors. The building's dimensions were 330 ft. by 420 ft. with 30 ft. by 30 ft. bays. The roof and ninth floor were constructed using conventional steel framing; however the eighth floor and those below were constructed using column trees. These trees were constructed of a 4-ft.-W24x61 stub girder shop welded to the columns. In the field W18x50 floor girders were connected to the stub girders using shear tab connections. Please see Figure 7, in Section 3.2 for a depictive representation. The W24x61 member is a Canadian shape that is similar to a W24x62 with a thinner web and shorter depth. This member size was originally produced by Algoma steel, but is now only made by Nucor-Yamato, and the Steel Deck Institute, because of lack of demand. The floor system was constructed of 4" lightweight concrete on 1-1/2" metal deck, and was attached to the floor beams and girders using shear connectors to create a composite floor system. All of the structural members were fire-protected with a sprayed-on mineral fiber that provided a 2-hour fire rating to the floors and a 3-hour rating to the columns (Barnett, 2002).

From the investigative report of this failure, it was determined that the shear tab connections failed due to secondary tensile forces developed from catenary action. However, in order to come to this conclusion and to provide a thorough understanding of the failure, a much more in depth analysis was performed. The analysis began with investigating the drop-span section of the column tree assembly for bending capacity and deflection as a function of time and temperature (calculated using the equations presented in Section 4.3.2). This same analysis was then done for the two alternative situations of continuous W24x61 and W18x50 girders replacing the column tree assembly.

The next step in the analysis was the investigation of the shear tab connections. These calculations followed those performed in *Appendix B: Structural Steel and Steel Connections* (Fisher, 2002) of the *World Trade Center Building Performance Study* (FEMA). These calculations were extended to analyze the bolted connections from 20°C (room temperature) to 650°C depending on the fire exposure used.

4.2.3 Alexis Nihon Plaza – Overview

The ten-story office building was constructed of a structural steel frame of 11 – 30 ft. by 30 ft. bays and 3 – 15 ft. by 30 ft. bays, as can be seen in Section 3.3, Figure 9. The girders (W24x76's), beams (W18x40's), columns and metal decking were all fire-protected with sprayed-on mineral fiber providing a 2 – 1/2 to 3 hour fire rating for the members. "The girder – to – column, beam – to – column, and beam – to – girder connections were made using double clip angles," which were "welded to the beams and bolted to the columns or girders," (Isner, 1986).

After the fire, the investigators found little to no distortion, such as bending, or twisting, of the steel members that had collapsed. This led to the conclusion that these members "were not exposed to excessively high fire temperatures or stresses," (Beitel, 2002). The collapse is believed to have occurred because the welds connecting the girders and columns failed causing the collapse of a 30 ft by 40 ft. section of the 11th floor.

To begin the analysis of this system, the girders material properties were calculated using the equations in Section 4.3.2. Using these values, RISA 2D, a structural analysis program was utilized to determine the end shear and moments in the girder. These numbers were then used for analysis of the weld shear capacity for a variety of angle sizes, weld thicknesses and strength. This analysis was also done for the bolts. Comparison of these values along with the capacity of the girder, led to the determination of the failure mode at various temperatures for various sizes of angles. These modes of failure are summarized and discussed further in Section 5.3.

4.2.4 One New York Plaza – Overview

One New York Plaza was a 50-story office building, "the first 20 stories are approximately 222 feet by 286 feet and the next 30 stories (the tower section) are approximately 143 feet by 286 feet," (Powers, 1970). The building was constructed of a reinforced concrete core with an outer structural steel frame and a composite floor system. The beams, girders, and columns were all protected with a sprayed-on asbestos fiber, which was later found to have not adhered properly to the steel due to rust. Fire which broke out on the 33rd and 34th floors caused shear connections to fail and beams to drop onto girder flanges, resulting in a partial collapse of the 34th floor.

This case was analyzed for the importance of the "lessons learned" that came from it. There were a number of recommendations made by the investigating committee that led both to a change in practice and means of failure in fires to come. These recommendations and their implications are discussed in Section 5.4.

4.3 Performance Investigation

4.3.1 Design Fires

Three design fires were used in the analysis for each of the cases studied. These fires simulated three different scenarios, one being a test scenario and the other two being natural fires.

The first design fire was the ASTM E-119 fire, which is a test fire used to rate the performance of building assemblies, such as ceilings and floor systems. There are a few key things to be noted associated with this fire. The first is that since this fire is simulated in a furnace, the tested assemblies are prototypes of the full-scale systems. The second is that "structural framing continuity, member interaction, restraint conditions, and applied load intensity" (Milke, 2002) are not applied while conducting tests under this fire exposure. While the test does not account for many factors like the few listed above, it is an internationally established test and is therefore useful for design comparisons. The gas temperatures from this fire were obtained from the ASTM E-119 time-temperature curve, as can be seen below in Figure 10.

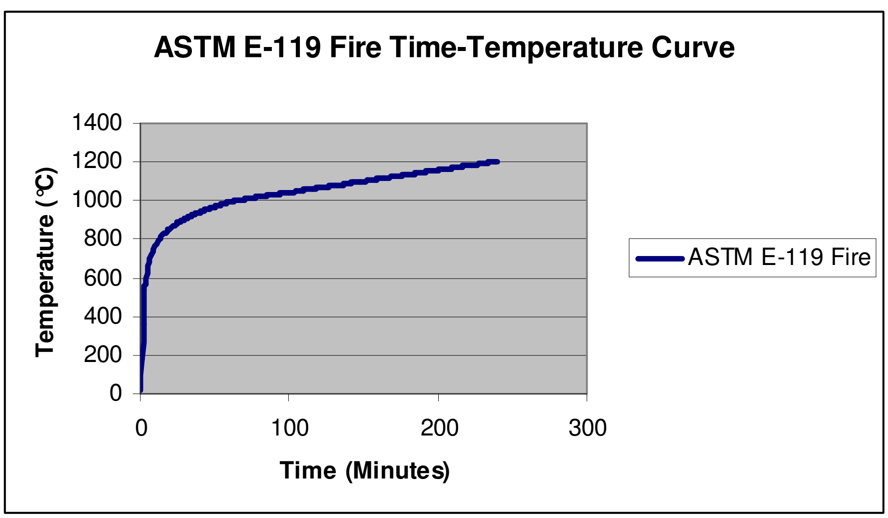

Figure 10 - ASTM E-119 Time-Temperature Curve

The second and third design fires used for analysis were a short duration-high intensity fire and a long duration-lower intensity fire. These fires were both defined by using a temperature-time relationship equation extracted from Section 4 of the *SFPE Handbook of Fire Protection Engineering*, which can be seen below in Equation 1.

$$T = 250\Gamma(10F)^{0.1/F^{0.3}} e^{-F^{2}t} + C\sqrt{\frac{600}{F}}$$

$$\text{where } \Gamma = 3(1-e^{-0.6t}) - (1-e^{-3t}) + 4(1-e^{-12t})$$

$$\begin{aligned} \mathrm{T} &= \text{fire temperature in}\ ^{o}C \\ \mathrm{t} &= \text{time in hours} \\ \mathrm{F} &= \text{opening factor in m}^{0.5} \\ \mathrm{C} &= \text{constant to account for boundary construction} \end{aligned}$$

Equation 1 - Expression for Defining Temperature Time Relationships

The equation above is used to determine the incline in natural fires. In order to define each of the two fires for use, different values had to be set to determine each. For the short duration-high intensity fire, the values for $F=0.12m^{0.5}$, C=1.0, and t varied from 0-0.5 hours. At t=0.5 hours, it was assumed that the fire decayed at a rate of 20°C per minute until returning to room temperature of 20°C. The short duration-high intensity fire curve can be seen below in Figure 11.

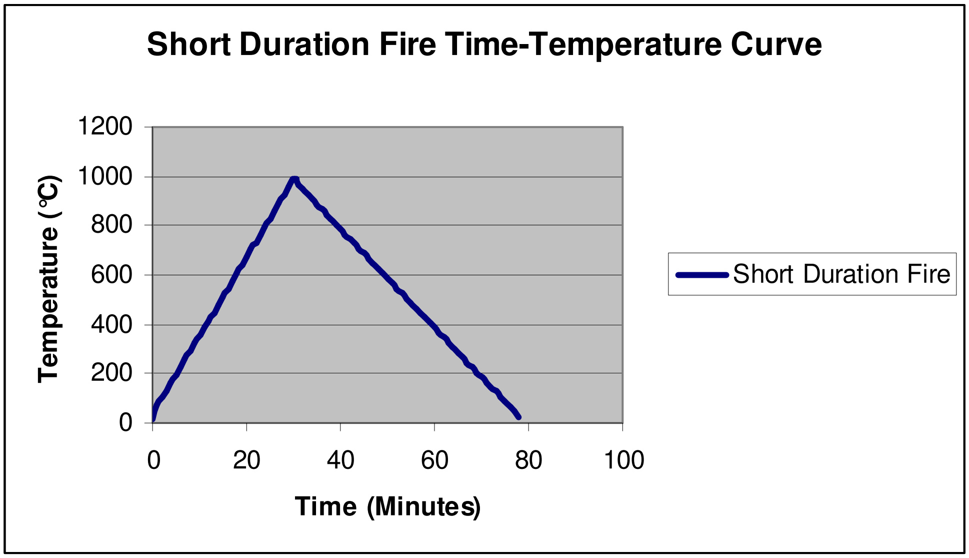

Figure 11 - Short Duration - High Intensity Time-Temperature Curve

The values used to define the long duration-lower intensity fire were F=0.04m$^{0.5}$, C=1.0, and t varied from 0-1.5 hours. At t=1.5 hours, it was assumed that the fire decayed at a rate of 10°C per minute until returning to room temperature of 20°C. The long duration-lower intensity fire curve can be seen below in Figure 12.

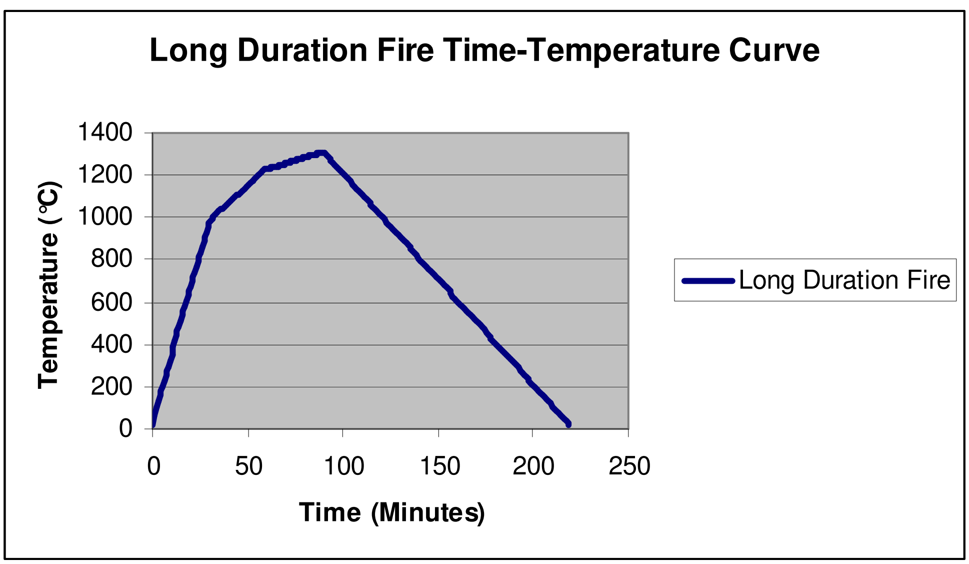

Figure 12 - Long Duration-Lower Intensity Time-Temperature Curve

4.3.2 Performance Calculations

Once the temperatures for each of the design fires were obtained, these values were then used to calculate the temperature of steel as a function of the gas temperatures and time by using a heat transfer analysis and the lumped mass approach. The lumped mass method assumes that the steel has little resistance to heat through conduction, and therefore the entire steel member is at the same temperature. The equation then used to acquire the change in steel temperature is seen below in Equation 2. This equation takes into consideration the heat capacity of the insulation as well as the steel as can be seen in Figure 13 below.

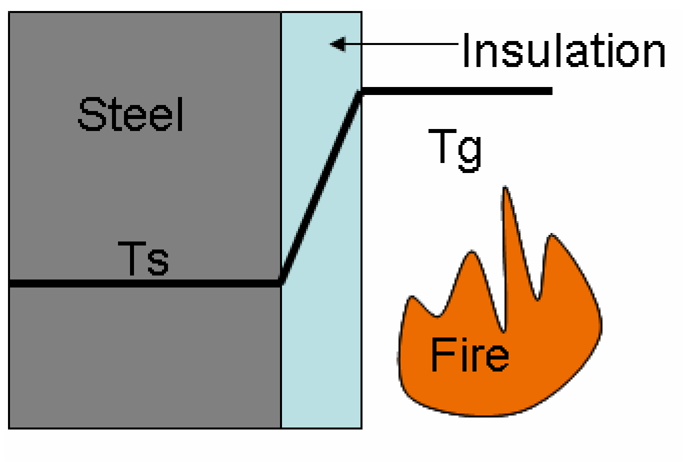

Figure 13 - Heat Transfer through Insulation

$$\Delta T_s = \frac{\frac{k_i}{t_i}}{\rho_s c_{ps}} \frac{A_i}{V_s} \left(\frac{1}{1+\xi} \right) (T_g - T_s) \Delta t - \frac{\Delta T_g}{1 + \frac{1}{\xi}}$$

Equation 2 - Change in Steel Temperature

Where zeta is a coefficient calculated using the equation in Equation 3 below.

$$\xi = \frac{\rho_i c_{pi} t_i}{2\rho_s c_{ps}} \frac{A_i}{V_s}$$

Equation 3 – Coefficient

The value of Ai/Vs was calculated using Equation 4 below.

$$\frac{A_i}{V_s} = \frac{(2d+3b_f-2t_w)}{A_s}$$

Equation 4 - Section Factor Equation

The time step factor Δt was calculated using Equation 5 below.

$$\Delta t \leq \frac{25000}{\frac{A_i}{V_s}}$$

Equation 5 - Time Step Factor

The other values supplemented into Equation 2 were taken from the tables below.

Material	Density kg/m^3	Specific Heat J/kg	Thermal Cond. k_i W/m
Sprays			
Sprayed Mineral Fiber	300	1200	0.12
Perlite or Vermiculite	350	1200	0.12
Boards			
Gypsum Plaster	800	1700	0.20
Compressed Fiber Boards			
Mineral wool, fibre silicate	150	1200	0.20

Table 3 - Thermal Properties of Insulation Materials (Buchanan, 2001).

Recommended Specific Heat Capacity [J/kg °C]	Reference
$c_{ps} = 520$	ECCS Technical Committee 3 – Fire Safety of Steel Structures
$c_{ps} = 425 + 0.773T_s - 0.00169T_s^2 + 2.22x10^{-6}T_s^3$ $20\ °C \le T_s \le 600\ °C$	*Eurocode 3*
$c_{ps} = 666 + 13002/(738 - T_s)$ $600°C < T_s \le 735°C$	
$c_{ps} = 545 + 17820/(T_s - 731)$ $735°C < T_s \le 900°C$	
$c_{ps} = 650$ $900°C < T_s \le 1200°C$	

Table 4 - Heat Capacities of Steel

For the analysis performed for each case, a spray-applied mineral fiber was used with a density of 300 kg/m^3 , a specific heat of 1200 J/kg°C, and a thermal conductivity of 0.12 W/m. The specific heat of steel was taken as 520 J/kg°C, and the density used was 7850 kg/m^3, which "remains essentially constant with temperature" (Buchanan, 2001). The calculated value for the change in steel temperature was then added to the previous temperature for steel to get the new steel temperature.

After the steel temperatures had been calculated for each fire condition, equations to obtain the yield strength and modulus of elasticity of the steel associated with those temperatures were used. The equations for these material properties are solely a function of temperature and do not account for any losses that may occur. They are taken from the *SFPE Handbook of Fire Protection Engineering* and can be seen below in Equations 6-9.

for $0 \le T \le 600C$

$$F_{yT} = \left(1.0 + \frac{T}{900 \ln\left(\frac{T}{1750}\right)}\right) F_{y0}$$

Equation 6 - Yield Strength for Temperatures between 0°C & 600°C

for $600C < T \le 1000C$

$$F_{yT} = \left(\frac{340 - 0.34T}{T - 240}\right) F_{y0}$$

Equation 7 - Yield Strength for Temperatures between 600°C & 1000°C

for $0 \le T \le 600C$

$$E_T = \left(1.0 + \frac{T}{2000 \ln\left(\frac{T}{1100}\right)}\right) E_0$$

Equation 8 – Modulus of Elasticity for Temperatures between 0°C & 600°C

for $600C < T \le 1000C$

$$E_T = \left(\frac{690 - 0.69T}{T - 53.5}\right) E_0$$

Equation 9 – Modulus of Elasticity for Temperatures between 600°C & 1000°C

The initial yield strength for these calculations, F_{y0}, was taken as 36 ksi, because when the cases being analyzed were built between the 1950's and 1970's; this was the predominant strength of steel used for construction. The initial modulus of elasticity, E_0

used was 29,000 ksi. The values calculated were then used as input for the RISA 2-D software analysis of the structural systems for the cases described above.

4.3.3 RISA 2-D

RISA 2-D Educational and Web-Demo are two versions of the RISA 2-D software package that allows for the design and analysis of structures. The two versions differ in the level of complexity of the information that can be input and the functions that can be performed. For this research project both versions were utilized to analyze the original and alternative structural systems for each of the cases studied.

The RISA 2-D Educational version was used to analyze structures by adjusting their modulus of elasticity values (calculated using the equations in Section 4.3.2) as a function of temperature. This is useful in determining deflection of members as a function of temperature. The approach above was also used in the RISA 2-D WebDemo version. However, by using this version, one could enter in offset distances as a part of the member boundary conditions to obtain reactions at the ends of the members that were used to analyze end connections.

The two versions of the RISA 2-D software were useful when analyzing different sections of each case study. The results from these analyses are discussed in Section 5 of this report, entitled Results.

5 Results

5.1 McCormick Place

The result of the structural failure at the McCormick Place Auditorium was due to the lack of fire protection on the structural steel trusses that supported the structure's roof. Without any fire protection, the steel was subjected to the extreme temperatures of a fire that resulted from an excessive fuel load. The temperatures reached in the building were unknown, but as can be seen from Figure 33 (Section 5.2.2), the critical temperature for most steel members is approximately 600°C and the trusses failed after only approximately one hour of exposure the fire (this temperature is marked with the thick orange line in the graphs below). Also the concrete in the lower level of the structure where the fire spread was severely spalled and spalling of concrete begins around 500°C.

When the building was constructed in 1960 in Chicago, Ill, fire protection of steel was not a mandatory design feature. The designers felt there was no need for fire protection systems because of the height (fifty feet) at which the trusses were located. It was assumed that a fire would not be large enough to substantially affect the integrity of the trusses at that height. Therefore, there was no insulation on the steel members and no installation of automatic sprinklers. As with the insulation, the sprinklers were omitted in the exhibition hall because of the height at which they were to be located was thought to be too high for sprinklers to be effective.

5.1.1 Parametric Study of Spray-Applied Insulation

Analyses were performed in order to determine if spray-on insulation would have had any positive effect on the resistance of the steel truss to the fire. This was accomplished by comparing the performance of the unprotected steel truss to that of steel trusses with varying thickness of spray-applied insulation. The insulation was varied from ½" to 1- ½", and the results were then plotted on the same graphs to enable comparison of the effects of the insulation on steel temperature over time. The yield strength of the steel (Fy) and the modulus of elasticity (E) of the steel during fire conditions were also investigated to illustrate the properties of steel under fire exposure.

The effects on the steel truss were modeled under the three fire conditions previously defined: the ASTM E-119 standard fire, a short duration-high intensity fire, and a long duration-lower intensity fire. These three fires each simulated a different scenario of fire intensity within McCormick Place and allowed for the analysis of the truss integrity under different scenarios.

5.1.2 Fire Exposure

The three graphs below illustrate the effect of insulation on steel temperature, yield strength and modulus of elasticity when exposed to the ASTM Standard E-119 fire.

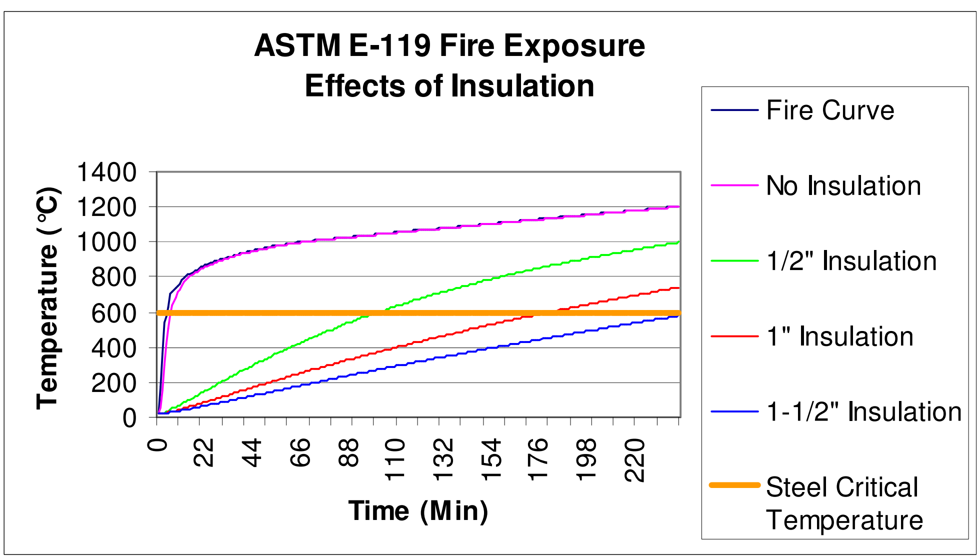

Figure 14 –Effect of Variable Insulation Thickness on Steel Temperatures

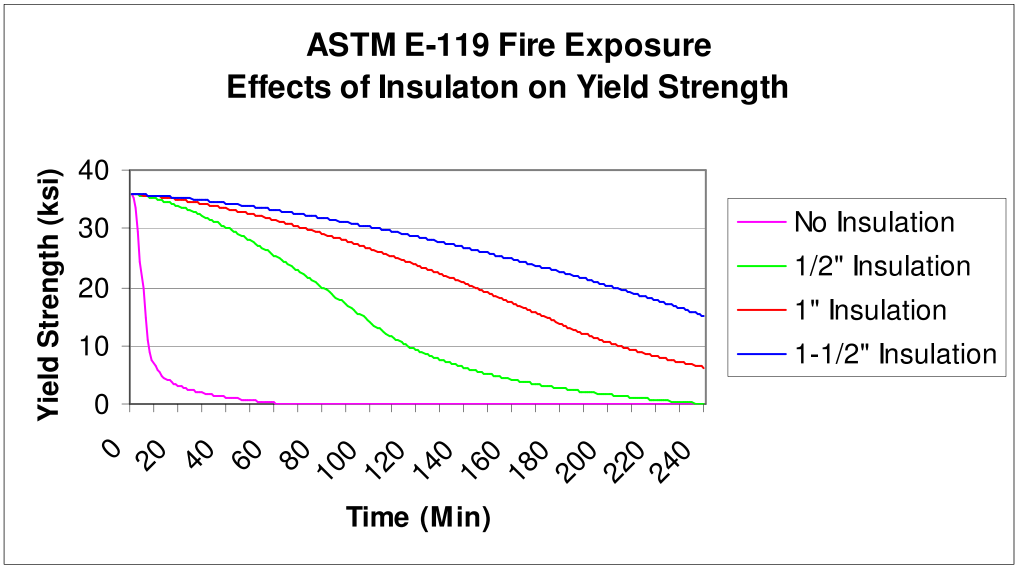

Figure 15 - Effects of Insulation Thickness on Yield Strength

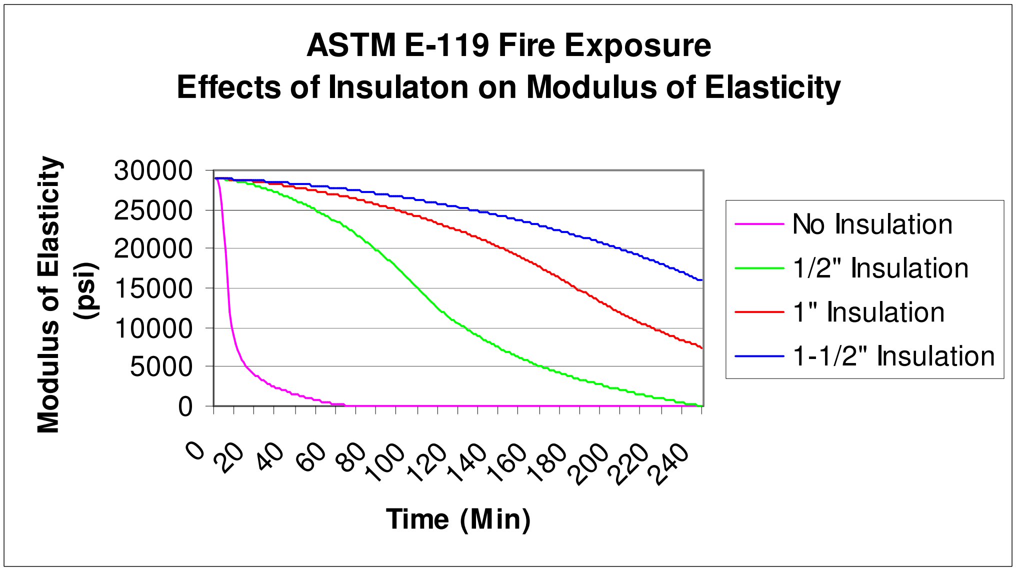

Figure 16 - Effects of Insulation Thickness on Modulus of Elasticity

The next three graphs show the effect of insulation on steel temperature, yield strength and modulus of elasticity when exposed to a short duration-high intensity fire.

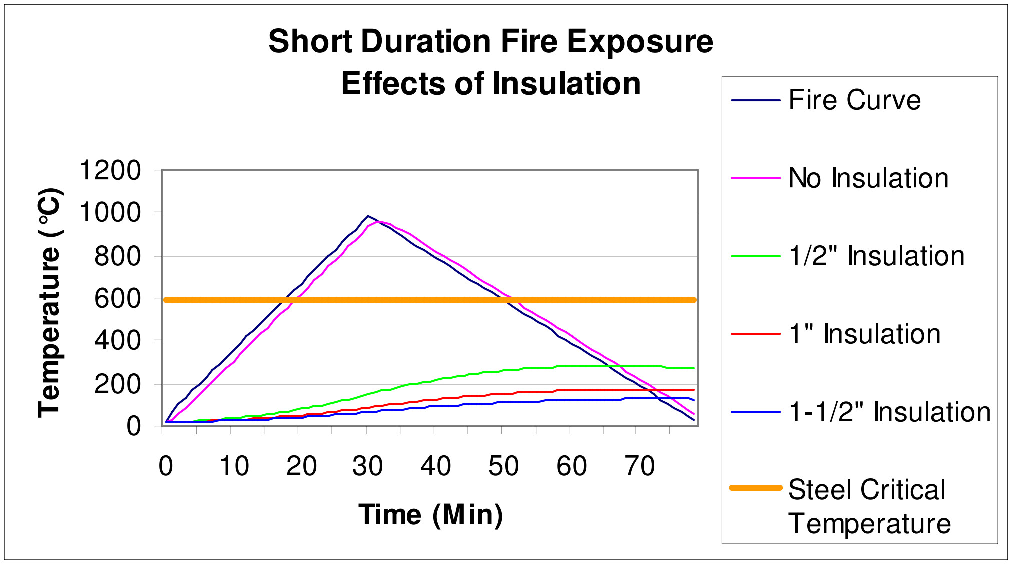

Figure 17 - Effect of Variable Insulation Thickness on Steel Temperatures

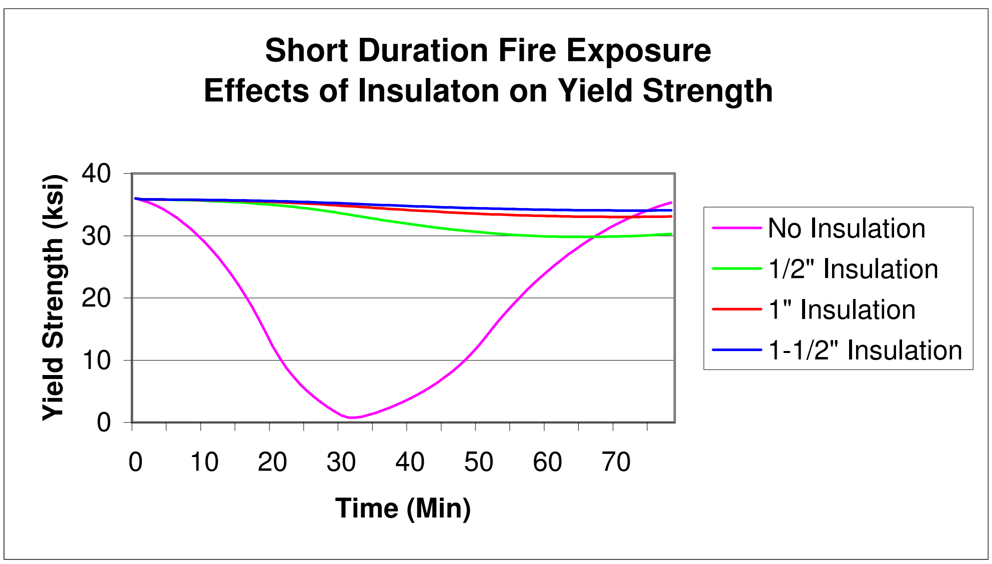

Figure 18 - Effects of Insulation on Yield Strength

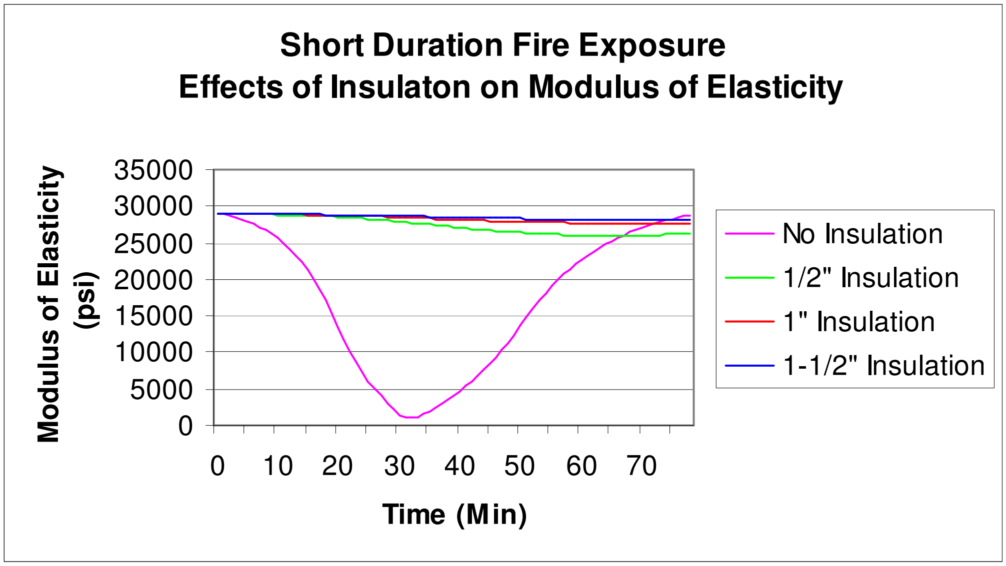

Figure 19 - Effects of Insulation on Modulus of Elasticity

The last three graphs show the effects of insulation on steel temperature, yield strength, and modulus of elasticity when exposed to a long duration-lower intensity fire.

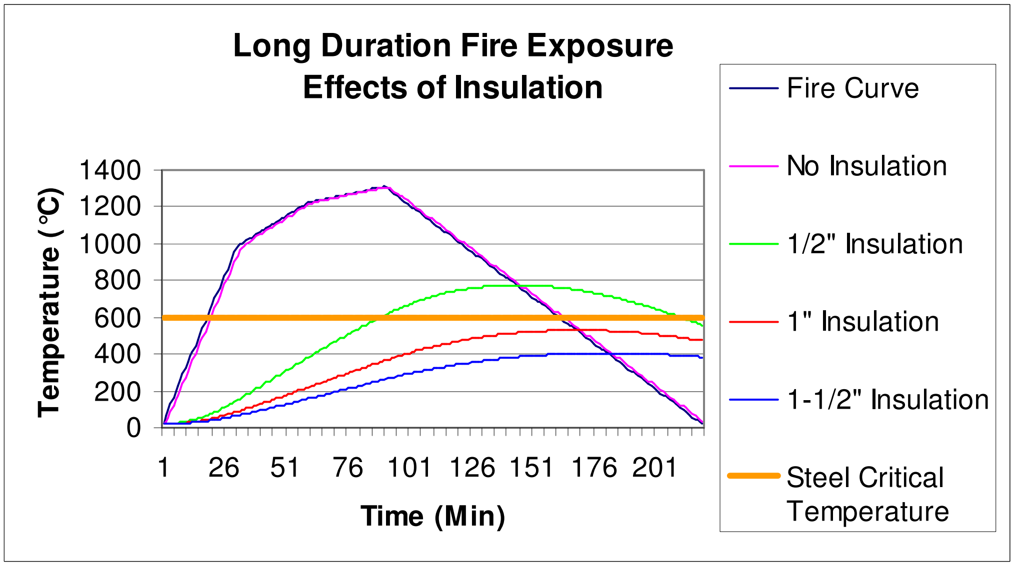

Figure 20 - Effect of Variable Insulation Thickness on Steel Temperatures

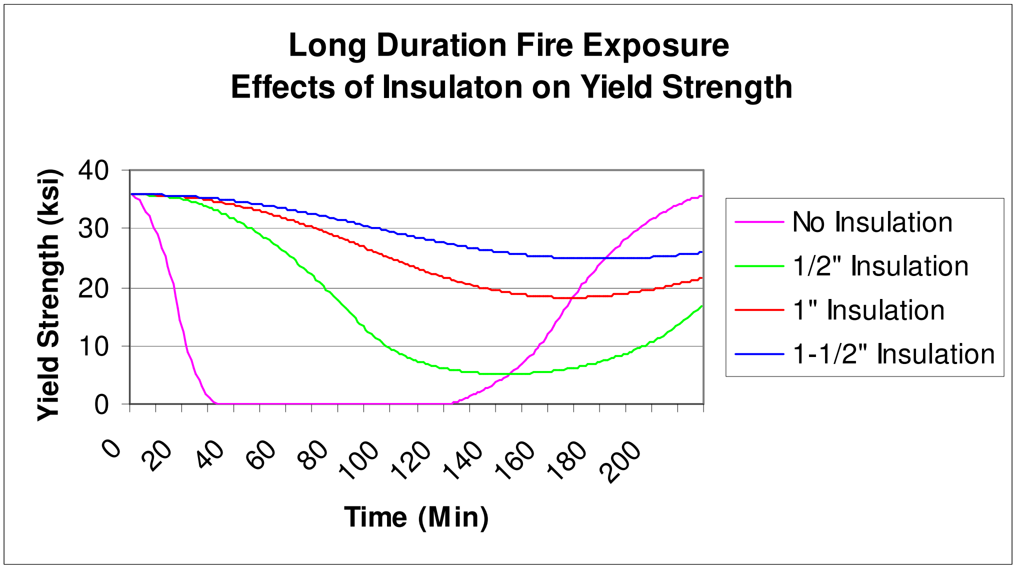

Figure 21 - Effects of Insulation on Yield Strength

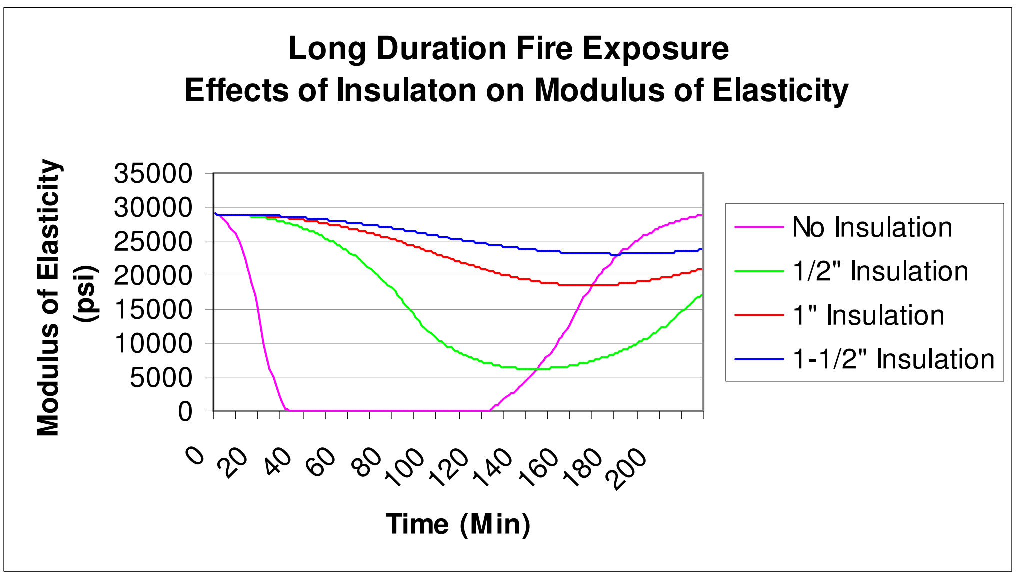

Figure 22 - Effects of Insulation on Modulus of Elasticity

As seen in the graphs, Figures 14, 17, and 20, above the use of any thickness of spray-on insulation would have reduced the temperatures of the steel truss significantly during all three design fires. In each case the greatest reduction of heat occurs early in the fire which is important because it may have given the fire department sufficient time to get the fire under control before excessive heating and structural instability occurred.

The yield strength (Fy) and modulus of elasticity (E) of the steel truss are important factors, because when these values are reduced, the strength of the steel is reduced, which could lead to failure. In all three cases, after one hour (approximate time of failure) with ½" insulation the loss of yield strength would have been 10 ksi as compared with the truss with no insulation that would have lost approximately all of its strength. Similarly, for the moduli of elasticity graphs, the loss for the ASTM E-119 and long duration fires is 5000 psi after one hour. For the short duration fire the loss is about 4000 psi for ½" insulation thickness. The loss for the cases with more insulation is less than that for the ½" insulation thickness. Under the short and long duration fire exposures one can see that the truss without insulation regains strength as the fire decays. This is true to a certain extent but unlike the graph depiction, it will never regain its full capacity. The graphs are off because the equations used to calculate these values only account for temperature and no other factors.

One difference to note between the short duration fire and the ASTM E-119 and long duration fires is the performance of the insulation thickness. Under the short duration fire exposure, there is little difference in the performance if insulation is provided. This is due to the fire peaking early and gradually cooling off. The short fire timeline enabled all of the insulation thickness to perform at approximately the same levels. Unlike the short duration fire, a greater thickness is more effective in the ASTM E-119 and long duration fires. The differences between 1" and 1½" of insulation are not as great as the difference between the ½" and 1" of insulation. Because of this, it would be a good decision to have at least 1" of insulation under these fire conditions.

The analysis performed in this investigation does not determine the mode of failure, however, it was discussed in the investigative report from observation that the trusses buckled in the center and a collapse of the roof ensued. This is the most probable situation since the compression members (bottom chord of the truss) would have heated the quickest and experienced warping and twisting due to the extreme temperatures produced by the fire. The center of the truss also did not have any diagonal members to support the top and bottom chord and therefore had the least resistance to failure as compared to the other pieces of the truss.

5.1.3 Automatic Sprinklers

After the 1968 fire, Underwriters' Laboratories (UL) conducted tests to investigate whether automatic sprinklers would have been effective in the exhibition hall of McCormick Place after the fire in 1968. These tests were conducted following the City of Chicago's building code criterion and a ceiling height of 50 feet was considered.

In order to recreate the fire, UL constructed booths similar to those located in the exhibition hall before the fire. Automatic sprinklers were then suspended 50 feet above the floor to simulate the location of the trusses relative to the fire. Under these conditions, it took six minutes from the start of the fire for the first of twelve sprinklers to activate. The automatic sprinklers quickly extinguished the fire, and the fire did not spread much further than the booth of origin. The maximum temperature experienced by the steel truss 50 feet above the ground was 510° Fahrenheit or 266°C.

These tests conducted by UL, clearly showed that the previous assumptions that automatic sprinklers would not be effective from higher distances were false. These tests cannot assume that the actual fire would have been extinguished as quickly as was accomplished by the tests, but they do provide evidence that automatic sprinklers would have reduced the extreme temperatures experienced and kept the fire under control. At the temperature of 266°C, the steel would have lost very little, if any of its material properties.

5.1.4 Discussion

The analysis done for this case was to determine if spray-applied insulation would have made a difference in the structural integrity of the steel truss. In order to determine this, the lack of insulation was compared to variable thickness of insulation under three different fire conditions. These values were calculated using the time-step calculation method previously discussed to obtain the steel temperature. The values of steel temperature were then used to obtain the material properties, yield strength and modulus of elasticity.

From the analysis it was determined that even a ½" spray-applied insulation would have greatly reduced the temperatures reached by the steel and would have allowed the truss to better maintain its structural integrity.

5.2 World Trade Center 5

The World Trade Center 5 case study served as the base case for the analysis performed in this project. There were several reasons for this, the first being that it was the most recent failure of the cases studied. It was also the case with the most known information and analysis as to why the failure occurred. The information provided in the technical report of the failure (Barnett, 2002), as well as in the appendices of the *World Trade Center Building Performance Study* (FEMA, 2002) report allowed the author to investigate alternative situations for the framing of the building to illustrate how steel performs when subjected to fire.

5.2.1 Floor Framing Analysis

As discussed in section 4.2.1, the original framing system for the World Trade Center 5 consisted of column tree assembly from the 4^{th} through the 8^{th} floor and conventional framing of the 9^{th} floor and roof, as shown in Figures 7 and 8 of Section 3.2. The steel beams and girders were topped with a lightweight concrete/metal deck floor to form a composite floor system. Performance under fire condition was modeled using the RISA-2D Educational software. In order to describe fire conditions for input into RISA-2D, time-step temperature calculations were done to determine the temperature of the steel with ½" insulation, 1" insulation, and 1-1/2" spray-applied mineral fiber insulation when exposed to the ASTM E-119 fire, a short duration-high intensity fire, as well as a long duration-lower intensity fire. Once the temperatures of the steel were calculated (at every 50° C, gas temperature), the yield strength and modulus of elasticity's that corresponded to those temperatures were calculated using equations from the *SFPE Handbook of Fire Protection Engineering*, samples of these calculations can be found in Appendix B. The moduli of elasticity's were then input into RISA to calculate the deflections associated with the increase in the temperature of the steel due to fire exposure. In order to approximate the behavior of the actual composite floor system, the moment of inertia for the drop-span girder (W18x50) was adjusted to be equal to the moment of inertia for the composite system. Also, the distributed load input to the program only reflected the live load to which the system would have been subjected, in order to account for shoring during construction, or any original camber that may have been present in the girder. Samples of these models can be seen in Appendix B.

The original model was then revised to replace the column tree system with (1) a W24x61 continuous girder and (2) a W18x50 continuous girder. The moments of inertia for both these beams was adjusted to reflect the moments of inertia of the composite systems, and both of these framing systems were exposed to the same loads seen by the original model. Both of these alternatives also used adjusted moduli of elasticity's to reflect temperature for the varying insulations, as done in the original analysis.

The deflections for each of these systems for exposure to the three different fire exposures previously mentioned can be seen in the figures 23, 24, and 25 below for ½" insulation thickness.

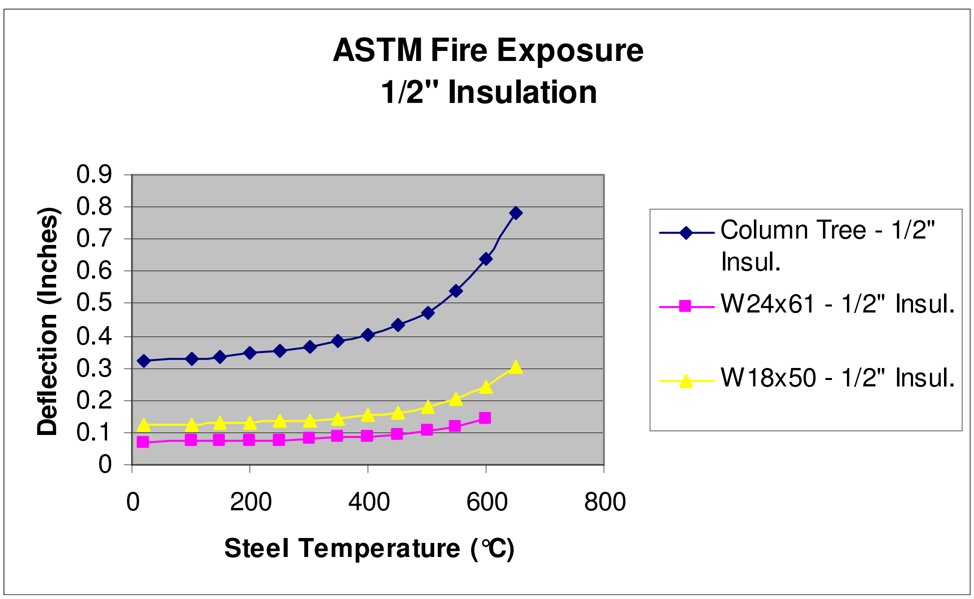

Figure 23 - Deflections under ASTM E-119 Fire Exposure

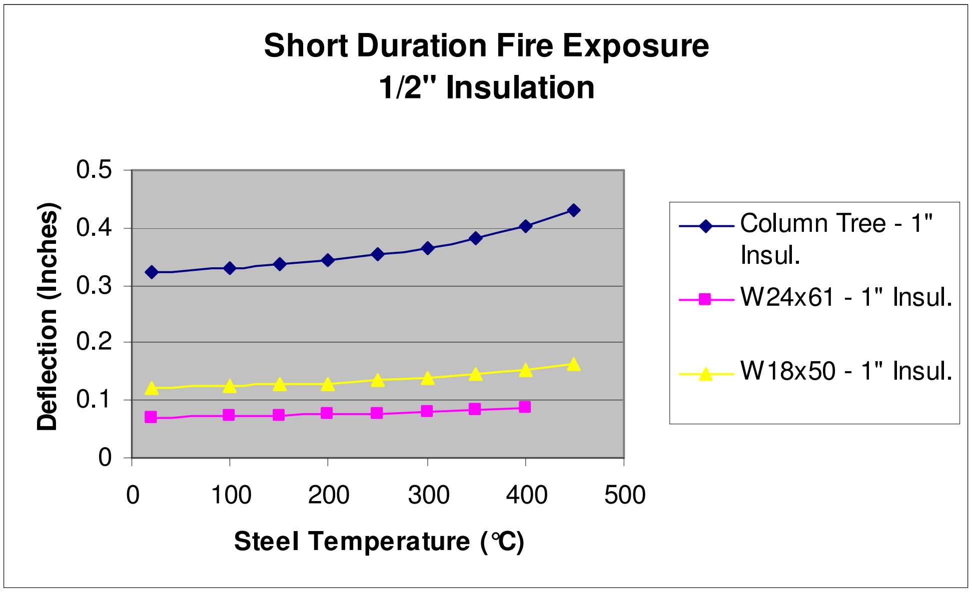

Figure 24 - Deflections under Short Duration-High Intensity Fire Exposure

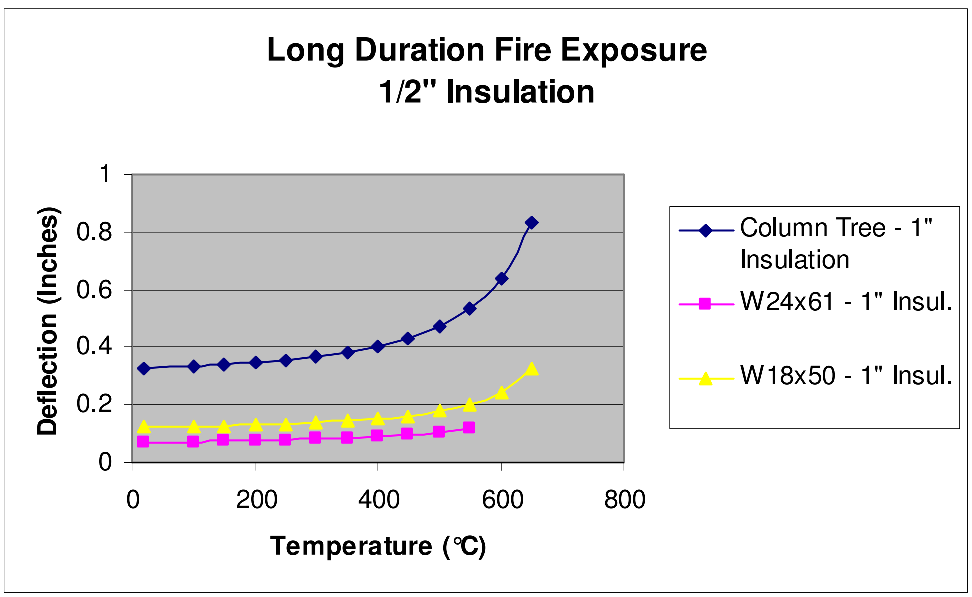

Figure 25 - Deflections under Long Duration-Lower Intensity Fire Exposure

The next three graphs show the deflections for each design fire using 1" insulation thickness.

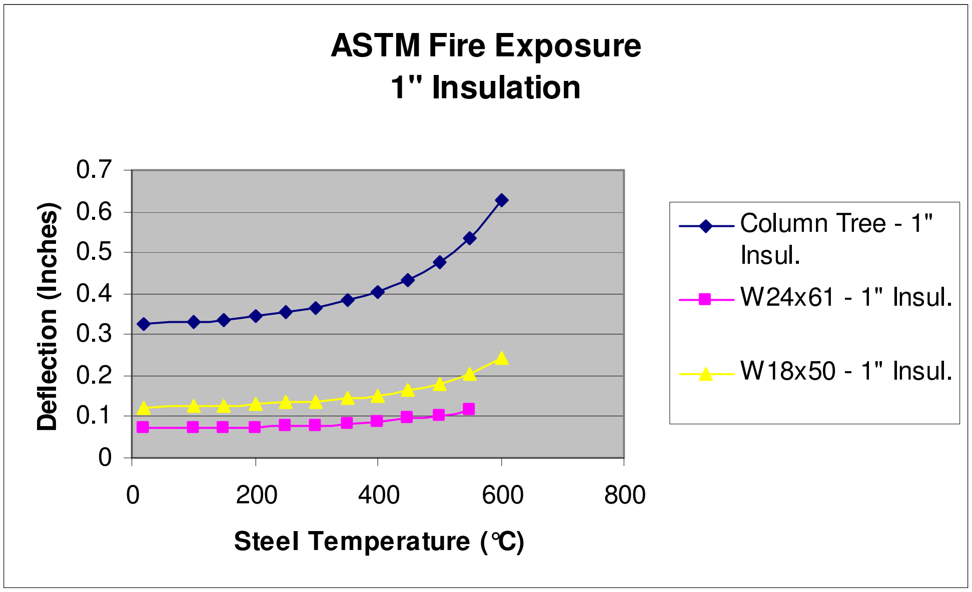

Figure 26 - Deflections under ASTM E-119 Fire Exposure

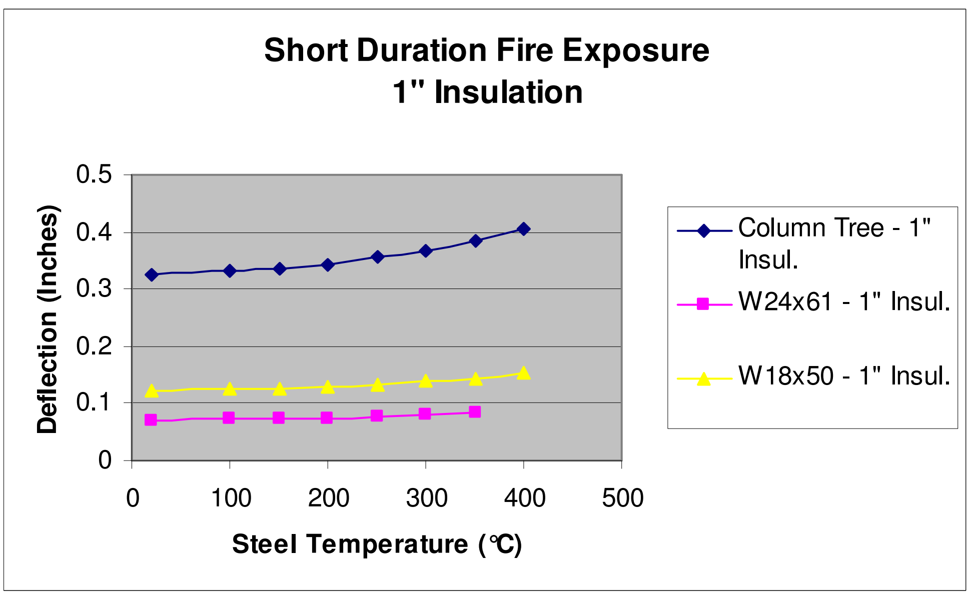

Figure 27 - Deflections under Short Duration-High Intensity Fire Exposure

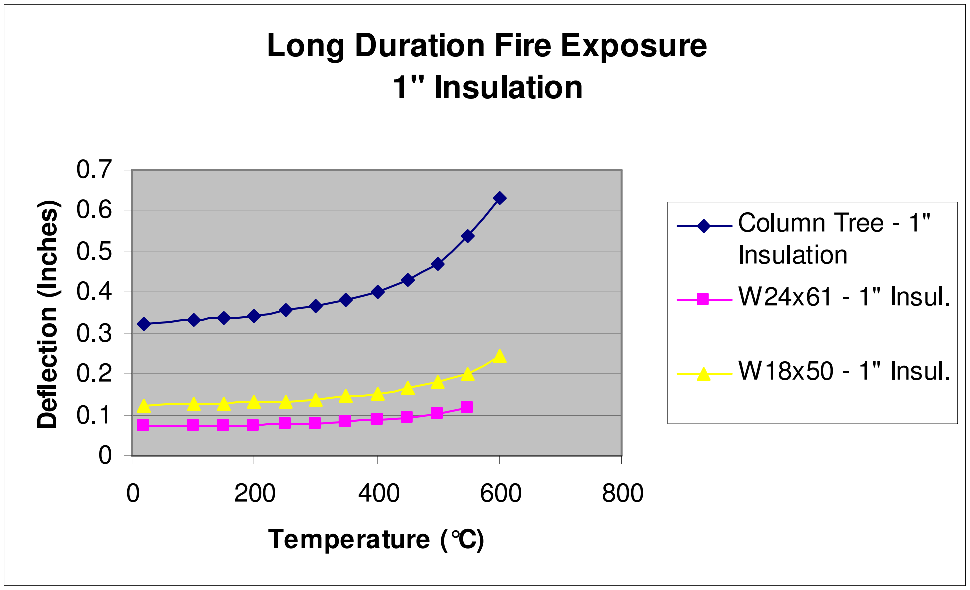

Figure 28 - Deflections under Long Duration-Lower Intensity Fire Exposure

The next three graphs show the deflection for the original and alternative framing systems with 1-1/2" insulation thickness, subjected to the three design fires.

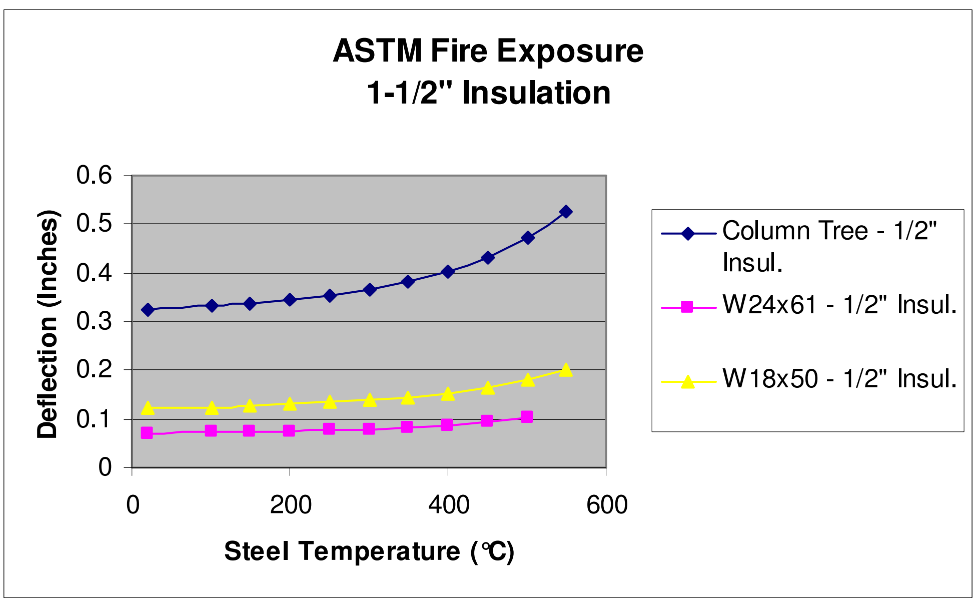

Figure 29 - Deflections under ASTM E-119 Fire Exposure

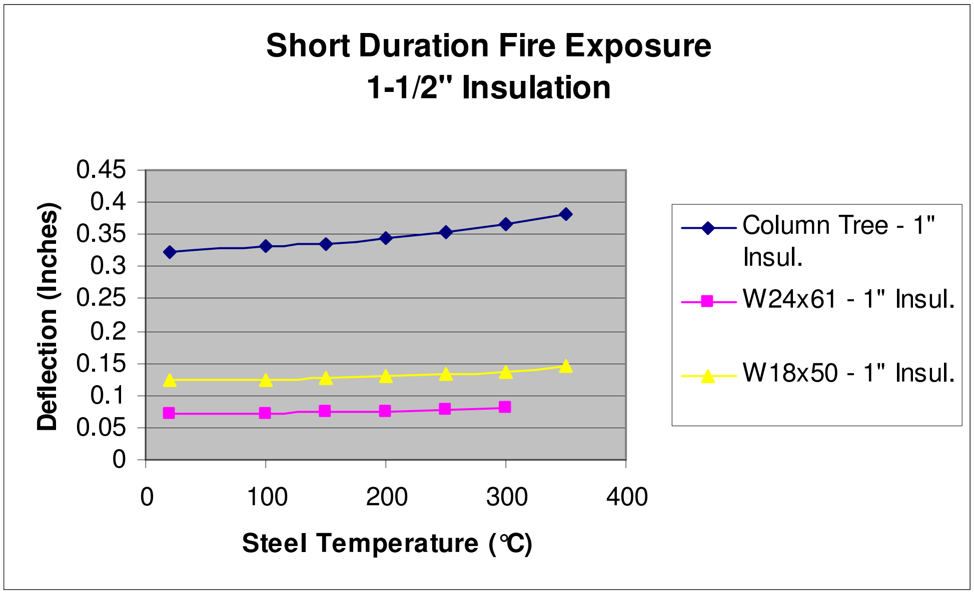

Figure 30 - Deflections under Short Duration-High Intensity Fire Exposure

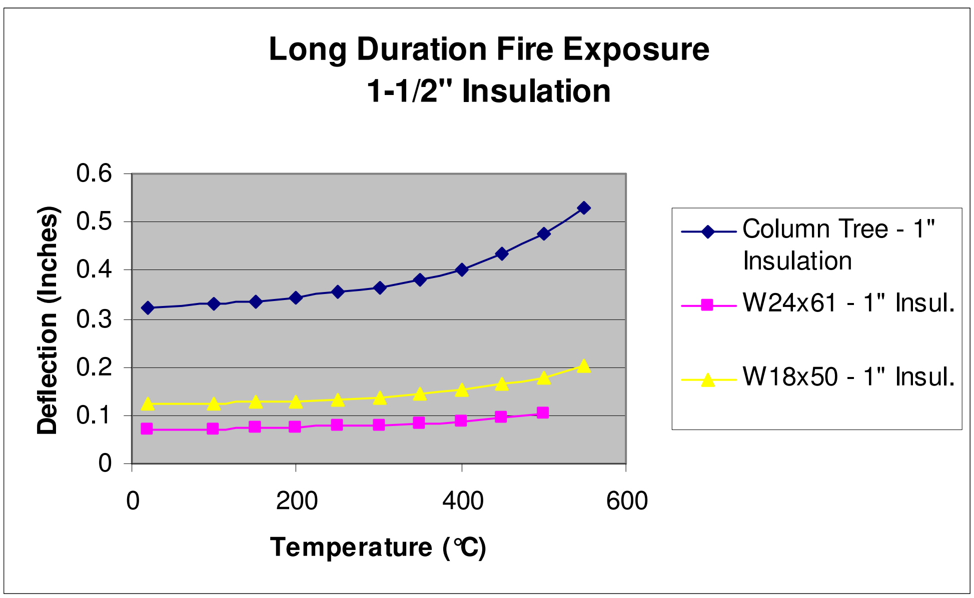

Figure 31 - Deflections under Long Duration-Lower Intensity Fire Exposure

For all three insulation thicknesses, one can see that the behavior under the ASTM E-119 curve and the long duration fire are very similar. It can also be seen that as the insulation thickness becomes greater, the deflection drops. An interesting point to note from these graphs is the similarity in behavior of the deflection patters, but the large difference in the amount of deflection between the continuous beams and the column tree assembly.

One should also note that the deflection limits of 0.73” (L/360) in the original case (L=22’) and 1” (L/360) for the two continuous span alternative cases (L=30’) is never exceeded under all three-fire conditions. This shows that all three framing systems could have theoretically handled the loads under the exposed fire temperatures so the system would not have failed due to deflection.

The technical report on this failure states “the structural damage due to the fires closely resembled that commonly observed in test assemblies exposed to the ASTM E119 Standard Fire Test,” (Barnett, 2002). It then goes on to say that the “local collapse appeared to have begun at the field connection where beams were connected to shop-fabricated beam stubs and column assemblies,” (Barnett, 2002).

5.2.2 Shear Tab Connection Analysis

The next step in the analysis process was to examine the shear tab connections where the failure appeared to have begun. In *Appendix B of the World Trade Center Building Performance Study* (Fisher, 2002), the three-bolt capacity, double shear capacity, and the tensile capacity of the shear tab connections were calculated at room temperature and at 550° Celsius. This was done by adjusting the yield strength values of the bolts by a factor determined from Figure 32 below; extracted from *Appendix A, Overview of Fire Protection in Buildings, WTC Building Performance Study.*

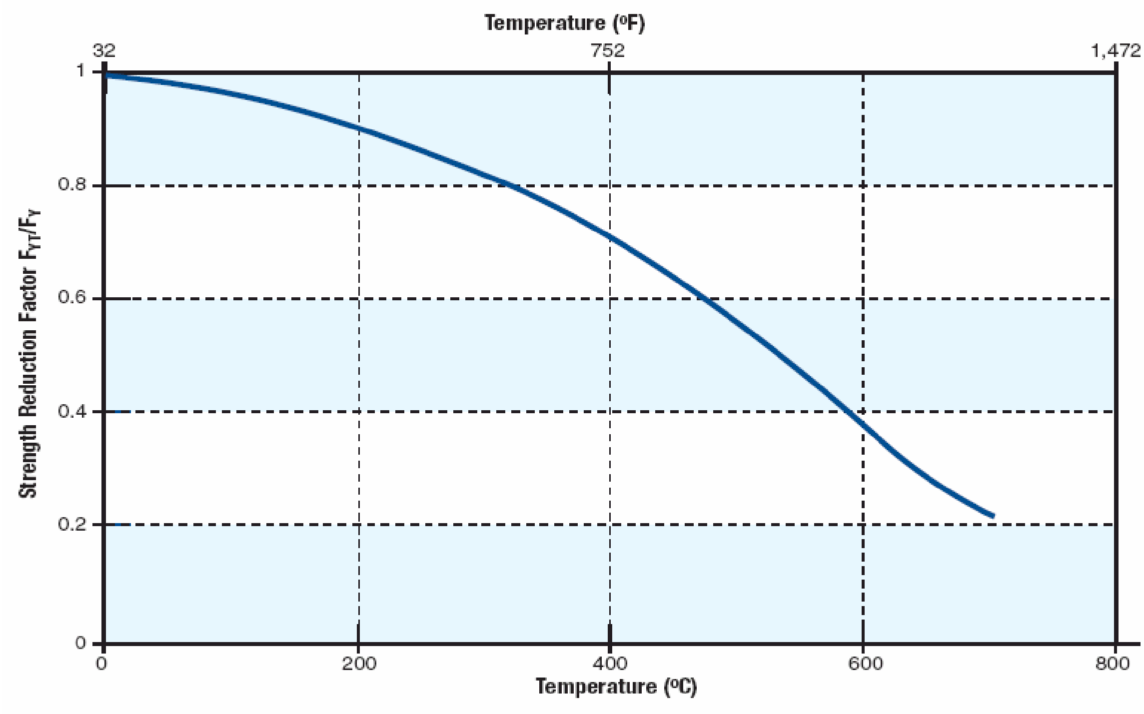

Figure 32 - Strength of Steel at Elevated Temperatures (qt. Milke, 2002)

For the purpose of this project, these calculations were extended to obtain the capacities of the shear tabs in the temperature range from 20°C (room temperature) to 700 °C. The critical temperature for steel beams/girders is listed in Figure 33 below is roughly 600 °C, which is taken to mean “the temperature where the steel has lost approximately 50 percent of its yield strength from that at room temperature,” (Milke, 2002) and it is a theoretical assumption that connections will degrade at the same rate or slower as the members they are connecting. If this is true then the connections should not have failed until about 600°C, and from the technical report it states failure at a temperature of 550°C or less.

Steel	Temperature
Columns	538 °C (1,000 °F)
Beams	593 °C (1,100 °F)
Open Web Steel Joists	593 °C (1,100 °F)
Reinforcing Steel	593 °C (1,100 °F)
Prestressing Steel	426 °C (800 °F)

Figure 33 - Critical Temperatures for Various Types of Steel (Milke, 2002)

Shown below, in Table 5, are the calculations for the three-bolt capacity of the connections, the 3-bolt shear capacity, double shear capacity, the tensile capacity, and the block shear capacity. While block shear was not a design requirement defined in the *1963 AISC Specification for the Design, Fabrication & Erection of Structural Steel Buildings,* it has been calculated for research purposes. The tabulated values for the double shear capacity are compared to the plastic shear capacity of the W24x61 girder and W18x50 girder and are greater; therefore the connections could handle the shear produced under the fire conditions. These shear values do not include the added shear that was produced from the collapsed floor weights. From Table 5 it can be seen that the governing mode of failure was from larger tension forces produced by catenary action than could be withstood by the shear tab connection. Since these forces are not present under normal conditions, they would not have been originally designed for. The second mode of failure seen from the table would have been block shear, which was also not a design requirement at the time it was designed. The capacity of the block shear at 200°C is less than the double shear capacity of the bolts and W24x61 girder at 550°C. In order to improve this, the thickness of the plate would have to double or a different bolt configuration could be tried.

		W24x61	W18x50	Shear	Shear	Tensile	Block Shear
Temperature	Strength Factor Ratio	Vp	Vp	Vu	Ru (D)	Ru	ϕRn
0	1	231.29	153.93	151.88	232.72	90.72	121.64
20	.99	220.98	153.36	150.36	230.39	89.81	120.42
100	.95	219.72	145.69	144.28	221.08	86.18	115.55
150	.93	215.10	142.62	141.24	216.43	84.37	113.12
200	.90	208.16	138.02	136.69	209.45	81.65	109.47
250	.85	196.59	130.36	129.09	197.81	77.11	103.39
300	.82	189.66	125.76	124.54	190.83	74.39	99.74
350	.77	178.09	118.09	116.94	179.79	69.85	93.66
400	.70	161.34	107.35	106.31	162.90	63.50	85.14
450	.65	150.35	99.68	98.72	151.27	50.97	79.06
500	.57	131.83	87.42	86.57	132.65	51.71	69.33
550	.48	111.02	73.61	72.90	111.71	43.55	58.38
600	.39	90.20	59.81	59.23	90.76	35.38	47.44
650	.29	67.07	44.47	44.04	67.49	26.31	35.27
700	.21	40.57	32.21	31.89	48.87	19.05	25.54

Table 5 - Steel Connection Capacities

5.2.3 Discussion

Catenary action is a phenomenon that occurs in composite beams as a result of thermal expansion in the member due to elevated temperatures. In all buildings, beams and girders have a certain amount of end restraint, even if simply supported. "The end restraints, although negligible at normal temperature, become significant at elevated temperature because of restraint to thermal expansion may cause enormous internal force and moment within the structural member," (Yu, 2005).

Catenary action was studied for the first time after the Cardington fire tests, conducted in 1995 on an 8-story composite steel frame. From these tests it was determined that when fire temperatures were below 400°C the slab acted as an extension of the compression flange of the structural steel and had little influence on the system, but when temperatures exceeded 500°C the slabs became a very influential part of the system. "The influence of membrane tensions in the slab cannot be ignored, particularly when a fire compartment is subject to high horizontal restraint from surrounding cool, stiff structure, or when it is vertically supported around its perimeter at protected lines of support. When the double-curvature deflections of floor slabs become large the influence of tensile membrane action can become very important in supporting the slab loading," (Huang, 2002). This tensile membrane action or catenary action can be useful in preventing a progressive collapse; however if the tensile forces developed are larger than the tensile capacities of the connections, the system will fail. This is the mode of failure that was believed to have occurred in the World Trade Center 5 column tree assemblies (Barnett, 2002). In Figure 34 below, one can see the catenary action developed in the upper floors that had typical structural framing.

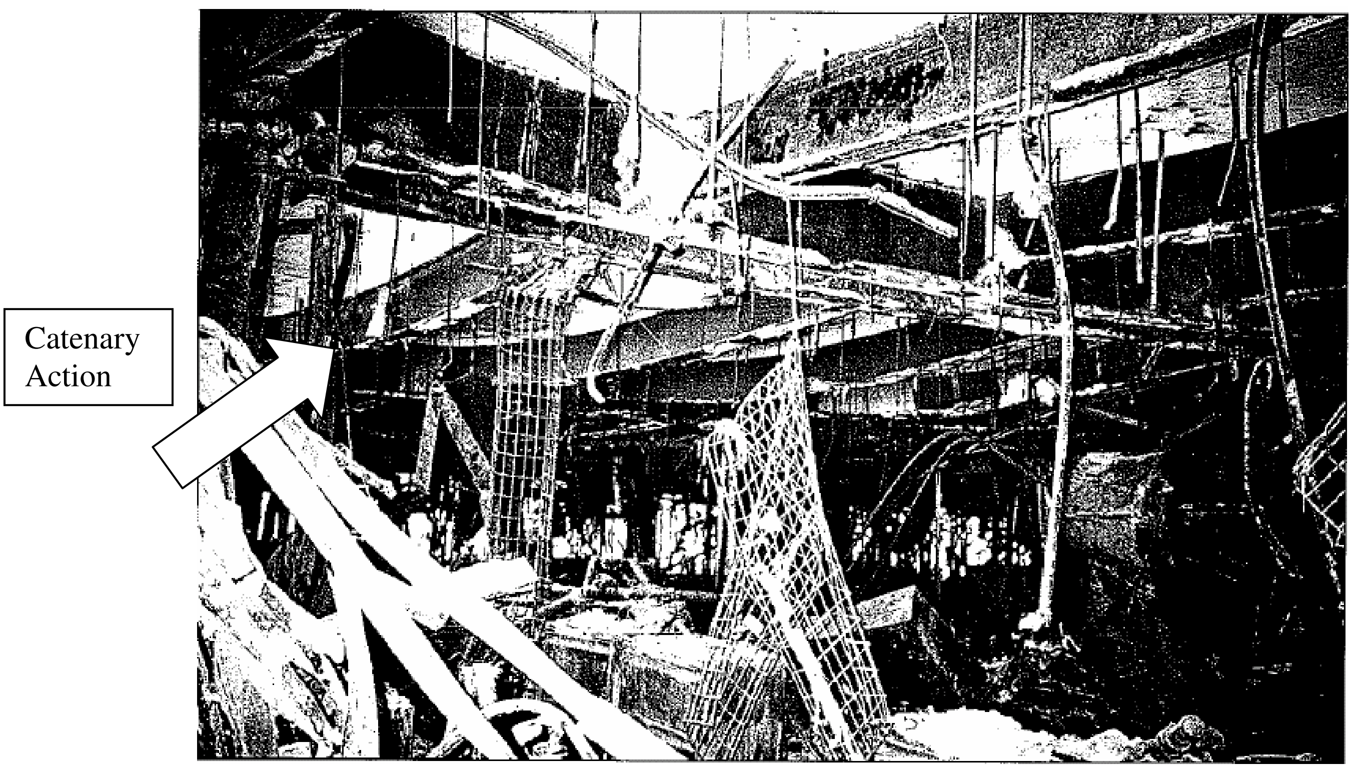

Figure 34 - Catenary Action in WTC 5

Since the destruction of the World Trade Center Plaza, there has been much research done to investigate this phenomenon. Currently, "tensile catenary action of floor framing members and their connections has been neither a design requirement nor a design consideration for most buildings," (Barnett, 2002). One paper entitled "Considering Catenary Action in Designing End-restrained Steel Beams in Fire" (Yu, 2005) provides simplifie equations needed to calculate the forces due to catenary action as a function of temperature for end restrained beams. Calculations using these equations were then compared to calculations done using the traditional design method to determine any advantages to using this approach. It was found that there was an advantage to using this new method when comparing beams with only axially restrained ends, but there was little advantage when the members had a fair amount of rotational end restraint (Huang, 2005).

Other studies have been done to determine the effectiveness of catenary action in preventing progressive collapses due to explosions. One study performed at the University of California, Berkeley, (Astaneh-Asl, 2006) tested the effectiveness of adding steel cables to floors, either during construction, or after, as a retrofit to prevent progressive collapse. The testing was performed on a one-story structural steel composite building constructed with a portion of the floor containing the steel cables and a portion without them. Each section was tested by removing one of the main columns to obtain the maximum loads experienced by the system. The section that did not have steel cables, was then retrofitted to have steel cables and the old connections were replaced. The same test that was performed on the other sections was then performed on this "new" section, and the loads were obtained. It was determined from these tests that if a building was designed with steel cables inside the slab, it could withstand load up to three times the design load; without cables, the system could withstand loads up to 1.2 times the design load; and the retrofitted section could withstand loads up to 1.5 times the design load. The conclusions from this study determined this design would be an economical way to prevent against progressive collapse (Astaneh-Asl).

While designing for catenary action was not a primary concern before September 11, 2001, it has become a very important topic. Just as there are active and passive techniques for fire protection, there are active and passive strategies for preventing against collapse. Active methods involve the capability to respond and to adapt to the stimulus, which would involve introducing sensors that can be monitored and send signals to mechanical devices, similar to those studied above with the addition of steel cables. Passive ways would be taking catenary action into consideration when designing, building in redundancies, etc… Either way, with the knowledge gained from the World Trade Center, catenary action should be taken into consideration during design.

One way to advance this study would be to gain a greater understanding of the catenary forces that were created, and compare those values with the tensile strengths of the plates. An alternative was to advance this study would be to alter the RISA model to incorporate column degradation as a function of time and temperature.

5.3 Alexis Nihon Plaza

As stated in Section 4.2.2, the believed cause of failure in this case was the welds that held the angle connecting the girder to the column flange. Investigation concluded that the section that collapsed was not exposed to excessively high temperatures from the fire (Isner, 1986).

5.3.1 Floor Framing Analysis

The floor framing system was comprised of W24x76 girders and W18x40 floor beams and a concrete composite floor. The girders were connected to the column flanges by way of clip angles that were bolted to the webs of the girders and welded to the column flanges. In order to examine all aspects of this failure, the original floor system was modeled using RISA-2D WebDemo version. This version was utilized because of its capabilities to enter offset distances, which were needed in order to analyze the behavior of the system with the eccentricities associated with the welded end conditions. The welded clip angles were modeled as springs, and the girder was offset 1.5" on each end to determine the moments and shears that would act through the bolted and welded portions of the connection. These values were then used to analyze the capacities of the bolts and welds for various angle sizes and thicknesses exposed to the three fire conditions previously determined.

5.3.2 Angle Connections

The information on the connections for this case was extracted from *High-Rise Office Building Fire, Alexis Nihon Plaza*, which was the investigative report of the failure. The report did not provide any data on the size of the angles, weld thickness and strength, or the number of bolts for each connection. Data on the connections could not be found in a wider search of the literature. To get a range of data for analysis, angles of various sizes were used to calculate the shear capacity of the welds and the tensile, shear, double shear, bearing, and block shear capacities of the bolts. The capacities are calculated based on geometry and material properties; they would be compared with the forces predicted by analyses. Weld capacity is calculated using the moment produced from the girder, and the length of weld, which changes with the size of the clip angle. The shear capacities of the welds, bolts, and girder can be seen in the graphs below. The bolts were calculated for an L8x8x1/2" angle and an L8x8x1/4" angle. The shear capacities of the welds were calculated for weld metal with both a 60 ksi and 70 ksi tensile strength. The steel temperatures were calculated for ½", ¾", 1", and 1-1/2" insulation thickness for the three fire exposures. The 3/4" insulation thickness is the thickness that was present in the original design. To illustrate the difference in shear capacities of the structural elements, the capacities of the welds, bolts and girder were placed on a single graph for each of the three design fires. The solid red line seen in these graphs is the end shear produced by the girder on the connections. The three graphs shown below in Figures 35-37 are for a ½" angles and a 7/16" fillet weld with a 60 ksi electrode. The 60-ksi weld metal was chosen because it produces the worst-case scenario.

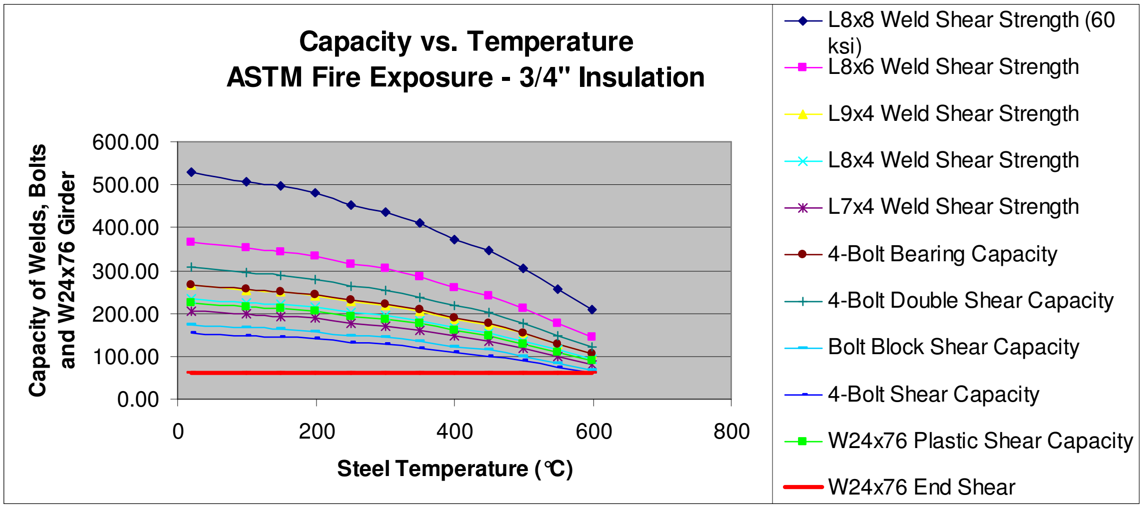

Figure 35 - Shear Capacities - L8x8x1/2" Angle & 7/16" fillet - 60 ksi weld metal

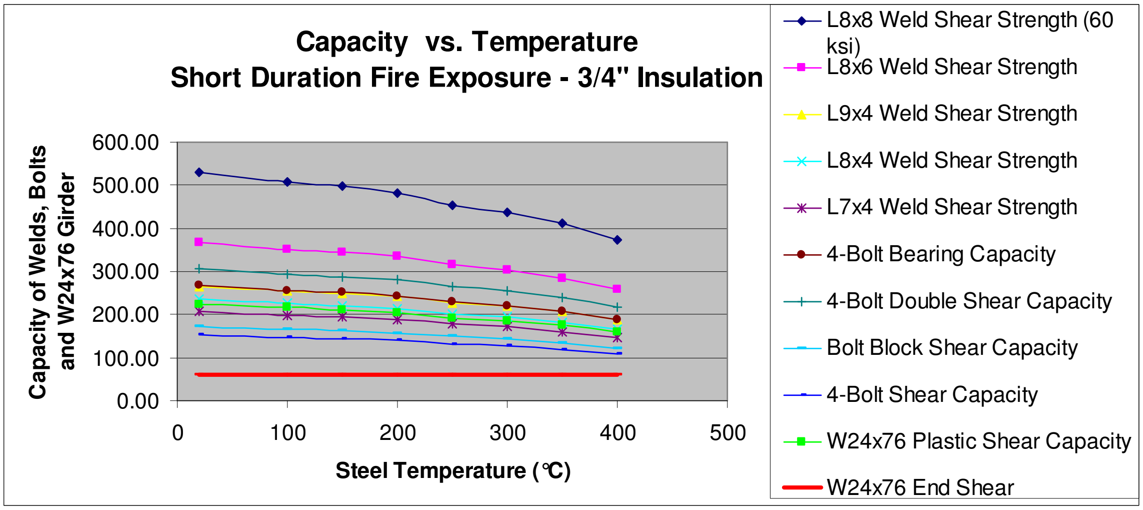

Figure 36 - Shear Capacities - L8x8x1/2" Angle & 7/16" fillet - 60 ksi weld metal

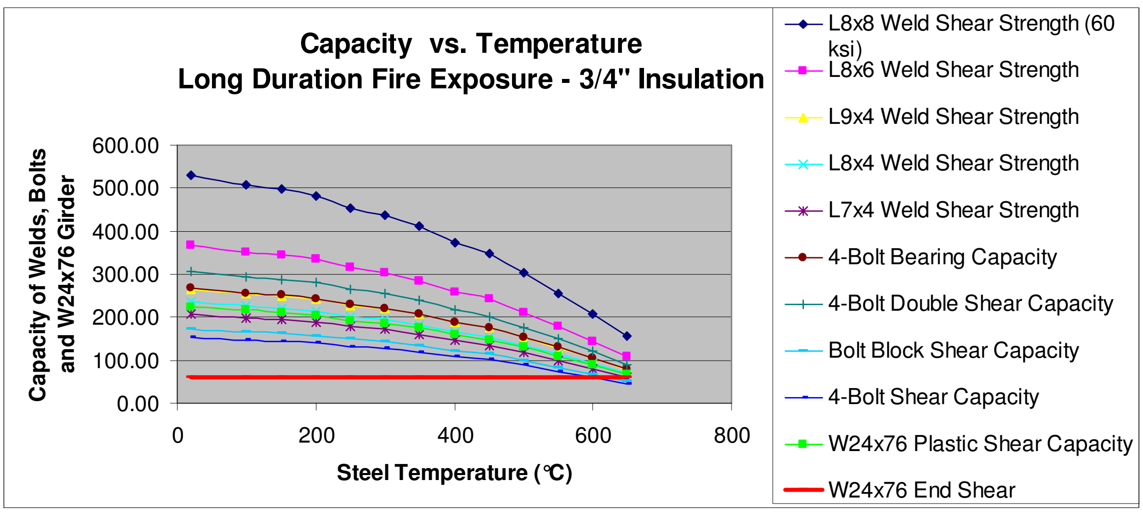

Figure 37 - Shear Capacities - L8x8x1/2" Angle & 7/16" fillet - 60 ksi weld metal

The three graphs shown below, Figures 38-40, illustrate the shear capacities for the elements listed above, for a variably sized ½" angles and a 1/4"-60 ksi weld.

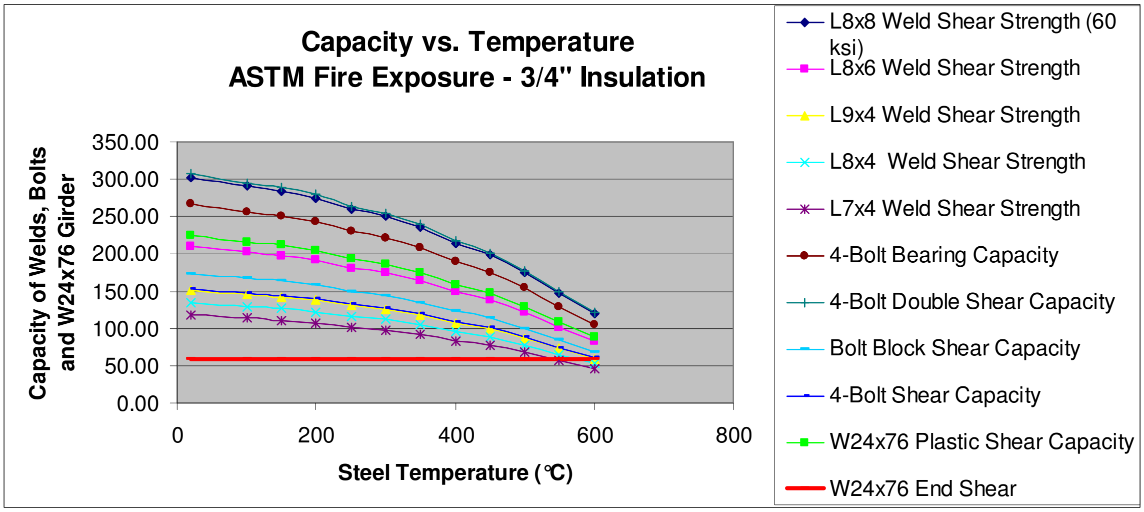

Figure 38 - Shear Capacities - L8x8x1/2" Angle & 1/4" fillet - 60 ksi weld metal

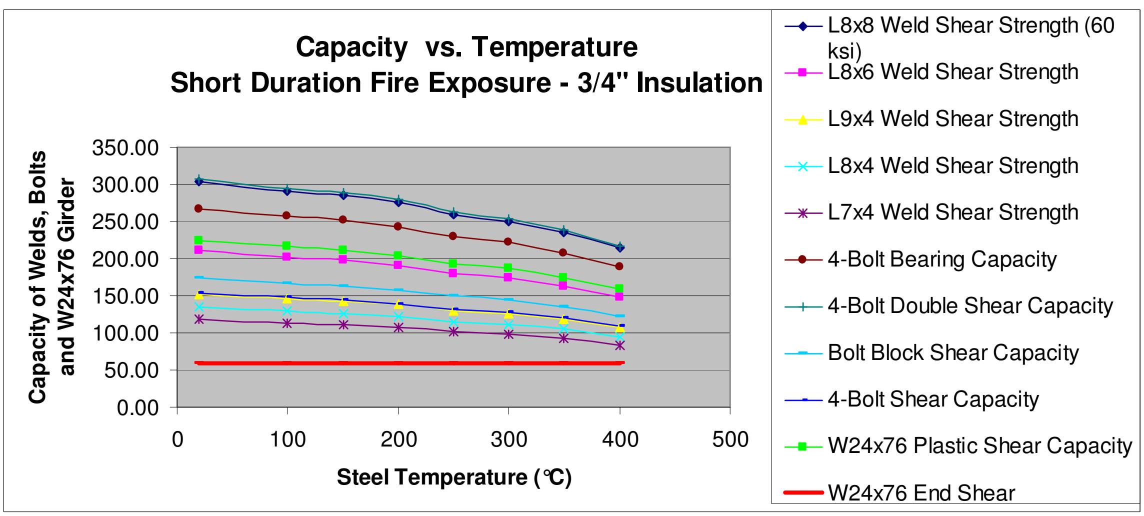

Figure 39 - Shear Capacities - L8x8x1/2" Angle & 1/4"fillet - 60 ksi weld metal

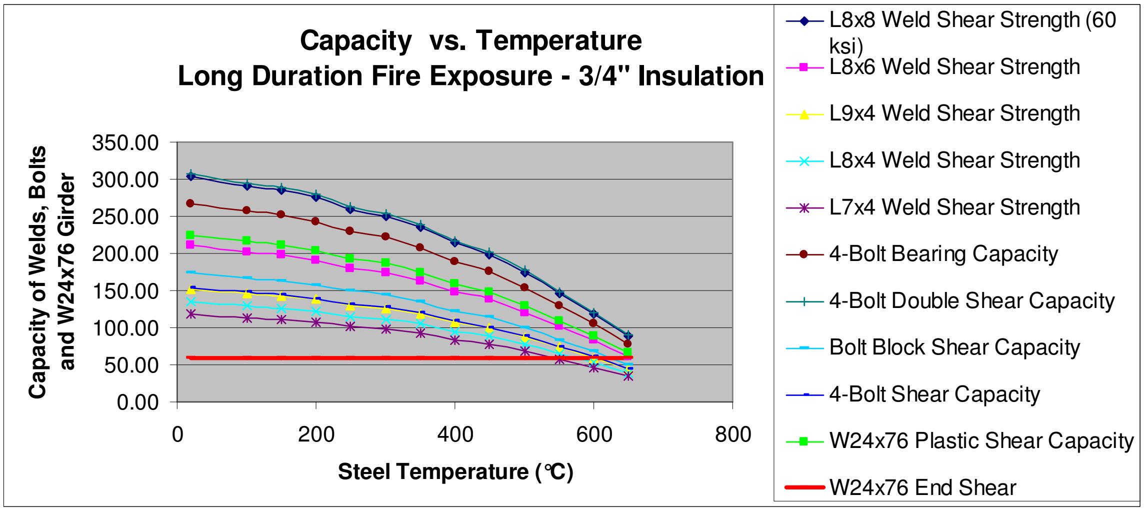

Figure 40 - Shear Capacities - L8x8x1/2" Angle & 1/4" fillet - 60 ksi weld metal

The three graphs below are the shear capacities of the structural elements for ¼" angles with a 3/16"- 60 ksi weld.

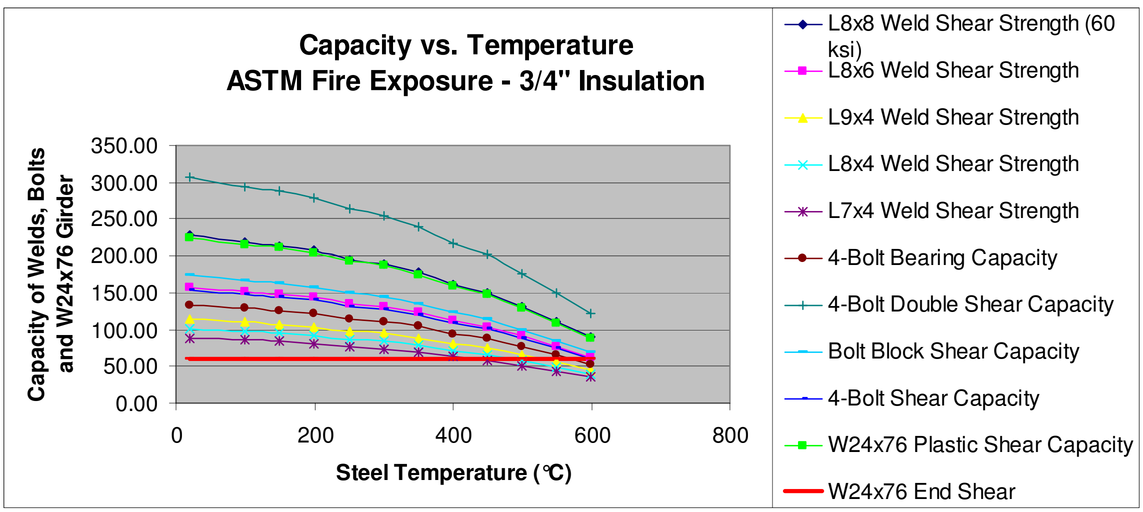

Figure 41 - Shear Capacities - L8x8x1/4" Angle & 3/16" fillet - 60 ksi weld metal

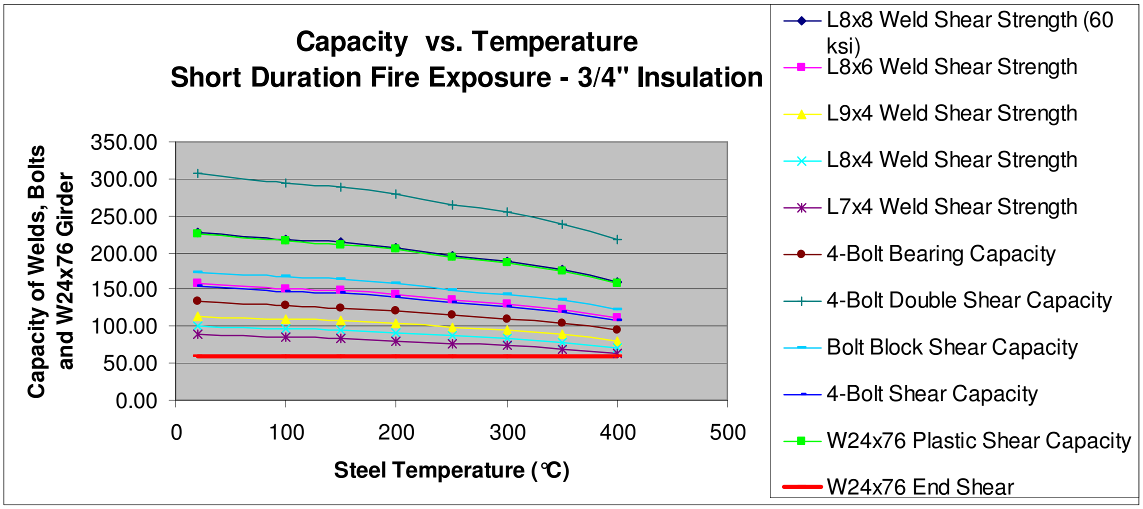

Figure 42 - Shear Capacities - L8x8x1/4" Angle & 3/16" fillet - 60 ksi weld metal

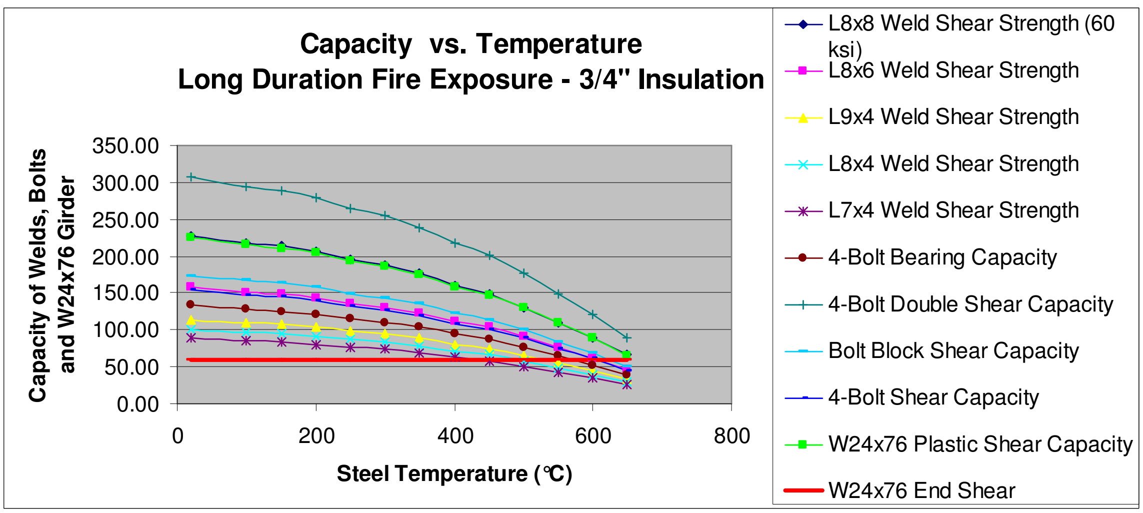

Figure 43 - Shear Capacities - L8x8x1/4" Angle & 3/16" fillet - 60 ksi weld metal

The final three graphs shown are the shear capacities of the structural elements for a ¼" angle and a 1/8"-60 ksi weld.

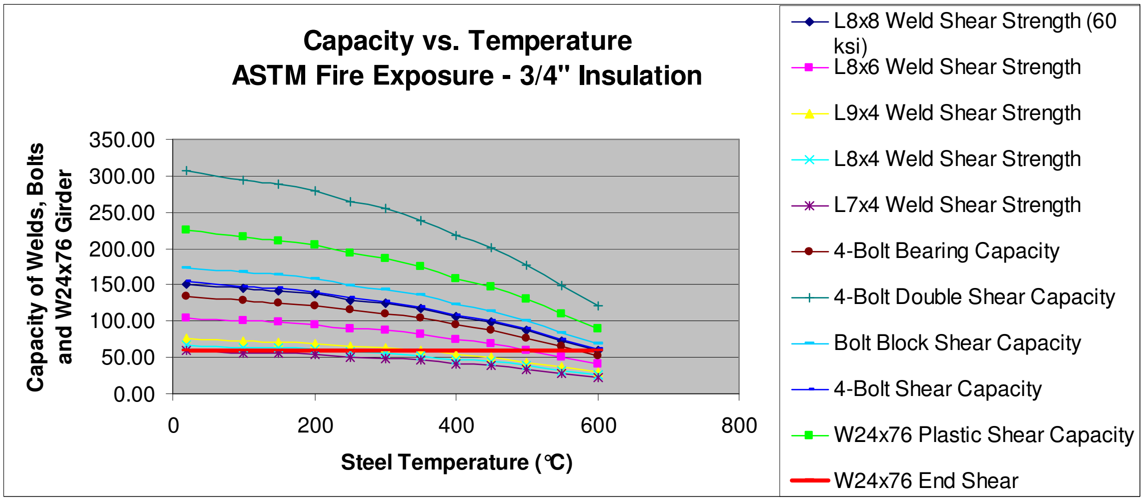

Figure 44 - Shear Capacities - L8x8x1/4" Angle & 1/8"fillet - 60 ksi weld metal

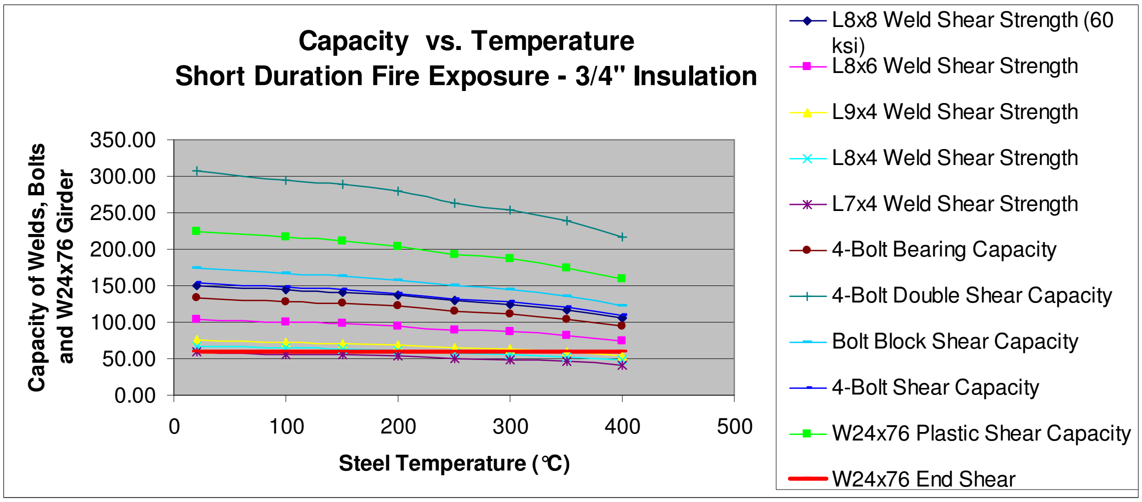

Figure 45- Shear Capacities - L8x8x1/4" Angle & 1/8" fillet - 60 ksi weld metal

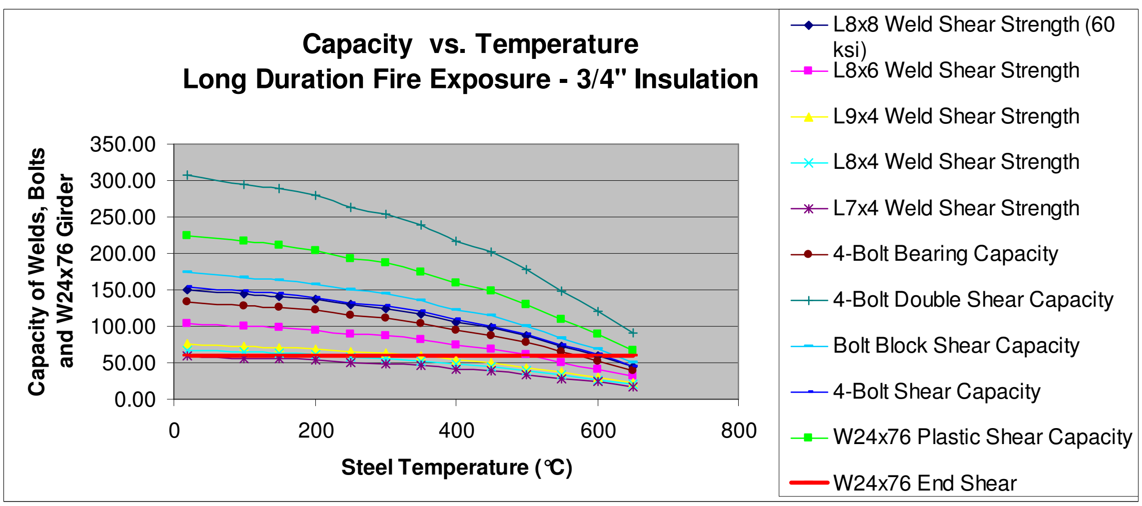

Figure 46 - Shear Capacities - L8x8x1/4" Angle & 1/8"fillet - 60 ksi weld metal

By looking at the twelve graphs above, the difference in strengths of variably sized ½” angles with 7/16” welds to variably sized ¼” angles with 1/8” welds is obvious. Below are tables summarizing the modes of failure by temperature for each of the cases illustrated.

Case # 1:

½" Angle – 7/16" Weld	600°C	625°C	650°C
ASTM	Bolt Shear		
Short			
Long	Bolt Shear	Block Shear	L7x4x1/2"

Case # 2:

½" Angle – ¼" Weld	575°C	585°C	600°C	610°C	650°C
ASTM	L7x4x1/2"	L8x4x1/2"	L8x8x1/2" L9x4x1/2"		
Short					
Long	L7x4x1/2"	L8x4x1/2"	L8x8x1/2" L9x4x1/2"	Block Shear	L8x6x1/2"

Case # 3:

¼" Angle 3/16 " Weld	425°C	533°C	500°C	575°C	600°C	625°C
ASTM	L7x4x1/4"	L9x4x1/4"	L8x4x1/4"	Bolt Bearing	L8x6x1/4" Bolt Shear	
Short						
Long	L7x4x1/4"	L9x4x1/4"	L8x4x1/4"	Bolt Bearing	L8x6x1/4" Bolt Shear	Block Shear

Case # 4:

¼ " Angle 1/8" Weld	50°C	200°C	320°C	500°C	585°C	600°C	625°C
ASTM	L7x4x1/4"	L8x4x1/4"	L9x4x1/4"	L8x6x1/4"	Bolt Bearing	L8x8x1/4" Bolt Shear	
Short	L7x4x1/4"	L8x4x1/4"	L9x4x1/4"				
Long	L7x4x1/4"	L8x4x1/4"	L9x4x1/4"	L8x6x1/4"	Bolt Bearing	L8x8x1/4" Bolt Shear	Block Shear

Table 6 - Failure Modes for Various Angles and Welds

Under normal conditions, all of the angles and welds investigated were adequate but as temperatures increased due to the three simulated fires, it is apparent how long each weld would effectively last. It is an assumption that connections degrade at the same rate as the members they connect; however, as can be seen above, this is not the case since the plastic shear capacity of the beam never reached the yielding end-shear strength of 59.18 kips.

The investigative report for this failure stated the:

"Girders and beams supporting the collapsed section of the floor slab were virtually straight and did not have the typical distortion such as bending, elongation, or twisting associated with steel frame members that have been exposed to excessive heat and high temperatures. In locations within the building, the welds that secured the clip angles to the columns broke, resulting in the failure of girder-to-column connections…" (Isner, 1986).

The reason for the welds breaking was not investigated through that report, but suggestions for the failure were offered, which included, effects of fire on welds, quality and adequacy of the welds, and the "possible effects of secondary and thermal stresses," (Isner, 1986).

Through the four cases illustrated above, the effect of fire on welds is clearly visible. Since the failure was attributed to breaking of the welds, cases two through four provide scenarios governed by failure of the end welds. It was stated that the floor system was not exposed to excessive temperatures, case #4 demonstrates that weld failure may occur at temperatures as low as 50°C.

5.3.3 Weld Quality

Weld quality is a major factor in terms of structural adequacy. There are many components that go into the welding process and if one of these is altered then the quality of the weld could suffer. There are six common defects when discussing weld quality, these being: "incomplete fusion, inadequate joint penetration, porosity, undercutting, inclusion of slag, and cracks," (Salmon, 1996).

Incomplete fusion, as the name implies, occurs when the two metals being welded do not fuse completely. Several reasons why this may occur is the condition of the surface of the base metal, insufficient weld current, and welding the two metals together too quickly. Inadequate joint penetration occurs when the weld does not penetrate to the depth specified. This can occur by improper design, rapid welding, lack of welding current or the use of too large an electrode. Porosity in a weld refers to when gas pockets form and are trapped in the weld. This defect occurs by using too high of a current or an arc length which is too long. "Undercutting means a groove melted into the base material adjacent to the toe of a weld and left unfilled by weld metal," (Salmon, 1996). This can occur by using too much current or too long of a welding arc. Slag inclusion is a defect that occurs when slag, a metal compound formed by the chemical reaction, is trapped by rapid cooling. "Cracks are perhaps the most harmful of weld defects," (Salmon, 1996). Cracks may occur in the weld, base metal, or both due to internal stresses. Un-uniform heating, and rapid cooling can also cause cracks in the weld (Salmon, 1996).

Most of these defects can be prevented by "establishing good welding procedures, use only pre-qualified welders, use qualified inspectors and have them present, use special inspection techniques when necessary," (Salmon, 1996). While proper practice and inspection are required under normal conditions to ensure good weld quality, the quality of welds under fire conditions has not been studied. Tests are required to determine the behavior of welds when exposed to fire. Until these are done, heat transfer analyses like the ones performed above or finite element analyses must serve as the basis of understanding welds under fire conditions.

5.3.4 Discussion

The investigative report for this fire established the reason for the failure but merely speculated on the reasons for its occurrence. Through the analysis of this case, several parameters were investigated to determine if changes could have been made to prevent the failure. These parameters included insulation thickness, angle size, weld thickness and weld strength.

From the analysis it was determined that the insulation thickness did not alter the results significantly. Changes in the angle size, weld thickness and strength however altered the outcomes greatly. As can be seen in Table 6 above, the ½" angle, with 7/16" weld, strength 60 ksi, would have failed by bolt shear at 600°C before any of the weld thicknesses tested failed. As the angle sizes decreased and the weld sizes decreased, failure of the welds became the governing mode of failure. As can be seen from Table 6, certain configurations of angles and welds could have contributed to failures at temperatures between 50°C and 320°C which are far less than the critical temperature of 600°C for steel beams.

The output from the analysis done for this case illustrates the potential for weld failure at relatively low temperatures. The prevailing notion is that beam failure would occur before connection failure due to the location and size of the connections. However, the results presented above disprove that notion. To get a more realistic picture of the shear forces and moments sustained by the welds, one could proceed with the RISA analysis and alter the strength of the columns with time as a function of temperature. One could also perform a finite element analysis for just the angles to determine the internal stresses in the member.

5.4 One New York Plaza

One New York Plaza was a case chosen to illustrate the "lessons learned" that were generated from this fire. While member collapses occurred due to the failing of end connections and lack of fireproofing, these were very localized and did not give way to a progressive collapse. The importance of this case is the recommendations that came from the report, some of which changed local law and practices, and some of which went unheeded and the consequences of such actions would be seen again.

There were fourteen recommendations that came from the investigative report of this fire. The recommendations made were for all buildings similar to One New York Plaza and not specific to just that building. The recommendations are listed below (Powers, 1970):

1. "The use of highly flammable foamed cushioning should be prohibited.
2. The total fire load of "fire resistive" buildings must be reduced or automatic sprinklers installed.
3. Wire (power and communication) in any part of an air conditioning system should be encased in metal conduit or ducts.
4. The protection of steel members in a really fire resistive building must be accomplished by materials that cannot be readily removed or damaged. It is apparent that sprayed fiber may not be universally applied to the proper thickness, that proper adhesion to steel may not take place and that the protection may be removed in many locations, such as at partitions, where ducts or wiring is run, and where clamps and brackets are attached.
5. Vertical flues in exterior walls between the skin and inner walls of partition should be cut off at each floor by a horizontal fire barrier with fire resistance equal to the floor.
6. Where openings through floors for air conditioning ducts are permitted, the duct should go directly to a non-combustible material in the duct passage.
7. Wiring connections through floors should be provided with thermal insulation to prevent transmission of heat thereby negating the fire resistance of the floor.
8. Air conditioning system should preferably be restricted to serving only one floor.
9. Automatic smoke detectors should be provided at each opening in the return air shaft unless the building is sprinklered.

10. Air supply for computer rooms should be from a location remote from the building air-conditioning intake and discharge, so that the computer rooms will not be subject to smoke from fires in other sections of the building.
11. Means should be provided for venting the building during a fire.
12. Elevator call buttons should not be of a type that will call and elevator to a floor because of heat, smoke or flames.
13. Prefire plans should be drawn up for all buildings. This should include procedures for notifying occupants, calling the fire department, routes of exiting form the various floors and protection of valuable equipment.
14. Special equipment for fire department use for operating windows, shutters, fans and elevators should be provided and a planned procedure for emergency operation of the air conditioning system should be formulated" (Powers, 1970).

In some respects, building design and construction has come a long way since this fire. Advancements in fire protection and compartmentalization have aided in the containment of fire. Buildings of a certain size and classification must now be equipped with automatic sprinklers by law. Also required by law are pre-fire plan evacuation routes, lighted exit signs, and other requirements to aid in preserving lives.

The three major recommendations that failed to initiate any change are numbers one, four and eight. While fire retardant material is more commonly used today, plush furniture and excess material continue to severely increase the fuel loads in buildings.

Due to cost, the means of insulating steel and air-conditioning a building have not changed. The issue of insulating steel in a manner that does not sacrifice the integrity of the steel if removed has been seen time and time again in the aftermath of One New York Plaza. Most recently are the cases of World Trade Center collapses. In the case of One New York Plaza, the steel beams were insulated with a spray-applied asbestos fiber, however due to rust on the original steel; the fiber did not adhere properly. Also, in places that the protection did adhere, it was removed for the installation of ducts, wiring, or other necessary work. The fire began and quickly spread through openings in the ceiling where the tiles had been removed for wiring. The combination of the missing ceiling tiles and the lack of insulation allowed excessive heat to penetrate the steel. The temperatures reached resulted in bending and twisting of the steel beams and shear failure of the end connections.

In the 1970's there was a large demand for sprinkler installation throughout high-rise buildings. Since then, it has been the trend to decrease the thickness of insulation if automatic sprinklers have been installed. However, the effectiveness of automatic sprinklers is not certain, as seen in WTC 5 when the sprinklers did not operate to control the fire. In many cases automatic sprinklers become inoperable, for example, when the metal heads fuse due to the heat generated by the fire or are designed incorrectly.

While it is more economical to reduce the insulation if adding automatic sprinklers, it may not be the best option from a structural standpoint. If the sprinklers are ineffective, then the structure is left exposed to the fire.

6 Discussion

An important note to make when researching case studies for this project was the lack of data available for many cases. As acknowledged by Dunn (1988) in Collapse of Burning Buildings: A Guide to Fireground Safety and Beitel and Iwankiw (2002) in *Analysis of Needs and Existing Capabilities for Full-Scale Fire Resistance Testing,* there is a lack of recorded data to accompany failures due to fire. In order to study and gain a comprehension of the subject through case studies, information regarding failures due to fire need to be more consistently recorded and assembled in some form that is available for study.

As can be seen in the 2005 AISC *Specification for Structural Steel Buildings, Appendix 4: Structural Design for Fire Condition,* the trend is towards structural engineers becoming responsible for the design of structures when exposed to fire conditions. Since this has not been a design requirement in the past, structural engineers need to be educated in methods in which to perform these analyses and their implications. This can be partly accomplished through the study of cases that deal with failures due to fire.

There are several assumptions that must be noted when studying this manual. The first, and perhaps most important, was the assumed mode of failure for each case. These were determined from the investigative reports of the failures studied.

Other assumptions made involved the heat transfer analyses. The analyses performed in this project to determine the steel temperatures and material properties (yield strength and modulus of elasticity) are based on the lumped mass heat transfer method. In order to use this approach, certain assumptions were made. The most important assumption is the temperature of the steel throughout the entire cross section is at the same temperature in order to simplify analysis. This is based on the fact that steel is a good conductor of heat; however a constant temperature throughout a member is not generally the case under real fire conditions. Other assumptions made in these analyses were the values for the material properties of the steel and insulation, which can be seen in Tables 3 and 4.

Secondary analyses of the connections in the World Trade Center 5 and Alexis Nihon Plaza cases involved the use of RISA-2D structural analysis software. This software was used to determine the forces in the members of the framing systems for each case. For input into this program, some information was known, but not all, which led to assumptions of the boundary conditions, material properties, and loading of the members. One major assumption made was that as the steel girders in the composite systems degraded the columns maintained their original properties. While columns often have a thicker fire protection layer and lag the degradation of the other members, they do still degrade to some extent with respect to fire.

In the case study manual developed through this project, two major points have arisen. The first is the importance of insulation, which includes but is not limited to, the insulation material, its application, and the quality control issues that arise with insulation. As was discussed in the recommendations from the One New York Plaza investigative report, the application of spray-applied insulation material needs to be controlled or inspected in a manner as to ensure effectiveness. This point was again raised in the FEMA World Trade Center Building Performance Study: Data Collection, Preliminary Observations, and Recommendations 30 years later.

The second point was the need for an in-depth study of the behavior of connections.

> "The performance of connections seem to often determine whether collapse is localized or leads to a progressive collapse. In the standard fire tests of structural members, the member to be tested is wedged into a massibe restraining frame. No connections are involved. The issue of connection performance under fire exposure is critical to understanding building performance and should be a subject of further research," (Milke, 2002).

Three of the four cases examined suffered collapses due to connection failure. In the analysis of the World Trade Center 5, it was determined that the shear tab connections failed due to extreme tensile and shear (due to collapse loads) forces acting on the connections. The design of simple shear connections would not have included analysis for the secondary tensile forces developed during the fire. The second case examined, Alexis Nihon Plaza, investigated the capacities of weld and bolts when exposed to fire. It was found from the technical report of this failure that the floor system was not exposed to excessive heat from the fire. Analysis of the weld and bolts for various plate thicknesses revealed that it is possible for a sound design of welds could produce failure at relatively low temperatures. If structural engineers are to begin designing for fire conditions, guidelines must be established for determining acceptable temperatures and standards for calculating thermal forces.

There are certain limitations that accompany the analyses performed for the cases above. The first is the simplified equations used in the lumped mass heat transfer method. These equations do not consider the spread of heat through the steel. The values for the temperature change of steel according to time and gas temperature (every 50°C) were then used to obtain the material properties of the steel. These equations, seen in Equations 6-9, are solely based on temperature, limited to 1000°C, and do not account for any losses that may occur in the steel. Since the critical temperature for most steel members is 593°C (Milke, 2002), determined from Figure 33, failure would likely occur before ever reaching 1000°C.

Another limitation encountered was the use of the RISA-2D software. The two versions of this software differ in the complexity of the input for each program. A more extensive knowledge of this program would have been useful in furthering analyses for the cases studied.

The last limitation encountered was the modes of failure determined from the investigative reports. Analyses were performed with the modes of failure in mind and geared towards those end results. If the failure modes had been undetermined, a much more broad investigation would have had to be completed to determine various modes of failure.

7 Conclusions

The goal of the case study manual developed through this project is to provide structural engineers with a resource for the understanding of several failures that occurred due to fire. The manual can be used as a teaching aid at the undergraduate and graduate levels. The intention is to illustrate what went wrong in the design of the cases analyzed and alternatives that could have produced different outcomes. The manual should stimulate discussion of the subject area and promote interest in studying the subject further.

Through the analyses presented in this paper, it has been demonstrated that it is possible to determine modes of failure for different framing systems using simplified equations and models. The results of these analyses allow for a better understanding of how different structural elements behave under various fire conditions. The analyses performed for this project are related to specific cases but can be applied generally to alternative cases, allowing for structural engineers to design for fire conditions using simplified models.

The *2005 AISC Specification for Structural Steel Buildings, Appendix 4: Structural Design for Fire Condition* defines several different fires and methods by which analysis can be accomplished. Since this is the first time this section has appeared in the Specifications, there is much room for expansion. Alternatively, by this section appearing in the specification, many questions are raised with its implications. Currently, the NFPA is responsible for the fired protection systems that appear in their published codes, so the issue of responsibility is an issue. Also, if structural engineers are to assume this responsibility, their fees must increase, and peer reviewers must also learn the new design analysis techniques. Another issue that arises with designing for fire is the amount of material, which will increase the overall project cost for structures. Over the years buildings have become lighter and lighter, and by introducing the idea of designing for fire goes against the standards of design and construction as presently practiced. In order to move performance based design forward, the questions raised above must be answered.

If these questions are answered and designing proceeds in this area, there is much room for the development of useful material. By developing the available tools further and expanding upon them, a monograph, similar to the CRSI: Reinforced Concrete Fire Resistance (CRSI, 1980) book could be developed. A book of this sort could contain equations and methods for analysis, state of the art designs and sample designs, and analysis and discussion of failure cases.

Another direction for future work would be the development of furnace tests that simulate natural fires to get a more realistic idea of the behavior of systems under these conditions. Connections should also be tested under both the ASTM E-119 and natural fire exposures to determine their behavior based on tests rather than assumptions.

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Appendix A

The Impact of Fire on Structural Steel
Case Study Manual

The Effects of Fire on Structural Steel

Case Study Manual

Rebecca Nacewicz

Abstract

This manual was designed to aid in the understanding of structural steel as a material and the manners in which it fails when exposed to fire. In order to accomplish this, four case studies of structural steel collapses due to fire were examined. Three of the four cases were studied to elaborate on the mode of failure as stated through their technical reports. These cases were then analyzed for alternative configurations to determine other modes of failure. The fourth case was chosen to highlight the recommendations that came from the technical report as "lessons learned" that should be kept in mind during design.

Table of Contents

1 Introduction

The goal of this case study manual is to provide structural engineers with a resource for the understanding of several failures that occurred due to fire. The manual can be used as a teaching aid at the undergraduate and graduate levels or for personal study. The intention is to illustrate what went wrong in the cases analyzed and alternatives that could have produced different outcomes. The manual should stimulate discussion of the subject area and promote interest in studying the subject further.

The use of case studies in education has proven a good supplement to the lecture based teaching approach. Investigation of subject matter through case studies illustrates how, when and why failures occur. Education through this method promotes individual learning and an opportunity for extensive research. This manual provides investigations of structural steel failures due to fire. This subject is becoming more important as the trend for structural engineers to become responsible for the structural integrity of buildings when exposed to fire is on the rise.

This is apparent in the 2005 AISC *Specification for Structural Steel Buildings,* which contains a new section detailing design for fire conditions. *Appendix 4: Structural Design for Fire Condition* defines several different fires and methods by which analysis can be accomplished. One of the methods, lumped heat capacity analysis, was used for investigation of the cases contained in this manual. This method uses a simple heat transfer analysis to determine the temperatures in the steel as a function of time and temperature. The investigative approaches used, descriptions of each case, and the results from the investigations are presented in the sections below.

2 Performance Investigation

2.1.1 Design Fires

Three design fires were used in the analysis for each of the cases studied. These fires simulated three different scenarios, one being a test scenario and the other two being natural fires.

The first design fire was the ASTM E-119 fire, which is a test fire used to rate the performance of building assemblies, such as ceilings and floor systems. There are a few key things to be noted associated with this fire. The first is that since this fire is simulated in a furnace, the tested assemblies are prototypes of the full-scale systems. The second is that "structural framing continuity, member interaction, restraint conditions, and applied load intensity" (Milke, 2002) are not applied while conducting tests under this fire exposure. While the test does not account for many factors like the few listed above, it is an internationally established test and is therefore useful for design comparisons. The gas temperatures from this fire were obtained from the ASTM E-119 time-temperature curve, as can be seen below in Figure 1.

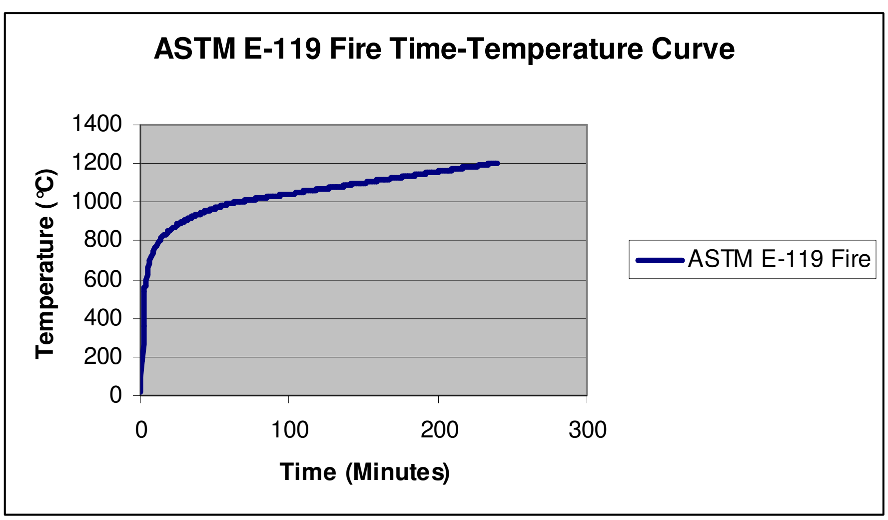

Figure 47 - ASTM E-119 Time-Temperature Curve

The second and third design fires used for analysis were a short duration-high intensity fire and a long duration-lower intensity fire. These fires were both defined by using a temperature-time relationship equation extracted from Section 4 of the *SFPE Handbook of Fire Protection Engineering*, which can be seen below in Equation 1.

$$T = 250\Gamma(10F)^{0.1/F^{0.3}} e^{-F^{2t}} + C\sqrt{\frac{600}{F}}$$

$$\text{where } \Gamma = 3(1-e^{-0.6t}) - (1-e^{-3t}) + 4(1-e^{-12t})$$

$\mathrm{T} = \text{fire temperature in } {}^{o}C$

$\mathrm{t} = \text{time in hours}$

$\mathrm{F} = \text{opening factor in } \mathrm{m}^{0.5}$

$\mathrm{C} = \text{constant to account for boundary construction}$

Equation 10 - Expression for Defining Temperature Time Relationships

The equation above is used to determine the incline in natural fires. In order to define each of the two fires for use, different values had to be set to determine each. For the short duration-high intensity fire, the values for F=0.12m$^{0.5}$, C=1.0, and t varied from 0-0.5 hours. At t=0.5 hours, it was assumed that the fire decayed at a rate of 20°C per minute until returning to room temperature of 20°C. The short duration-high intensity fire curve can be seen below in Figure 2.

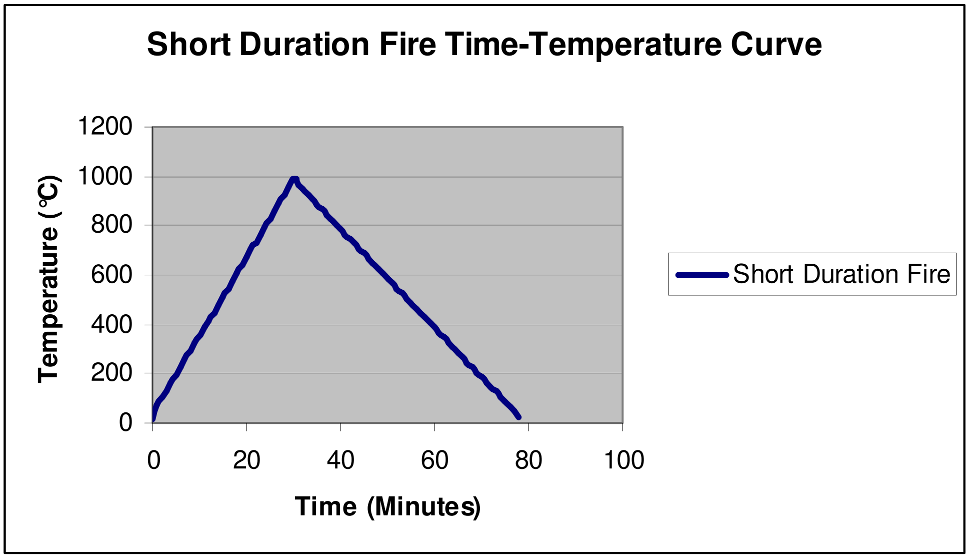

Figure 48 - Short Duration - High Intensity Time-Temperature Curve

The values used to define the long duration-lower intensity fire were F=0.04m$^{0.5}$, C=1.0, and t varied from 0-1.5 hours. At t=1.5 hours, it was assumed that the fire decayed at a rate of 10°C per minute until returning to room temperature of 20°C. The long duration-lower intensity fire curve can be seen below in Figure 3.

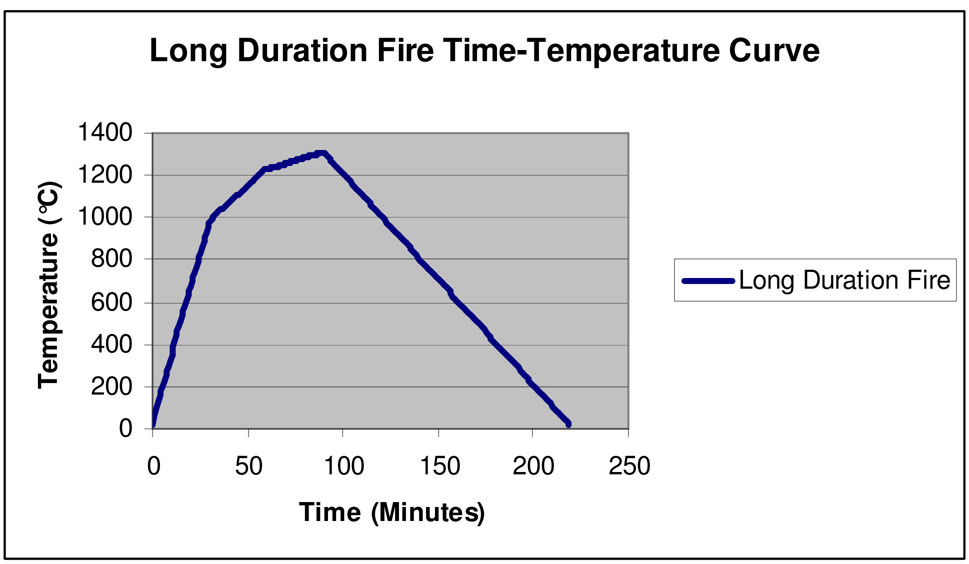

Figure 49 - Long Duration-Lower Intensity Time-Temperature Curve

2.1.2 Performance Calculations

Once the temperatures for each of the design fires were obtained, these values were then used to calculate the temperature of steel as a function of the gas temperatures and time at every 50°C by using a heat transfer analysis and the lumped mass approach. The lumped mass method assumes that the steel has little resistance to heat through conduction, and therefore the entire steel member is at the same temperature. The equation then used to acquire the change in steel temperature is seen below in Equation 2. This equation takes into consideration the heat capacity of the insulation as well as the steel as can be seen in Figure 4 below.

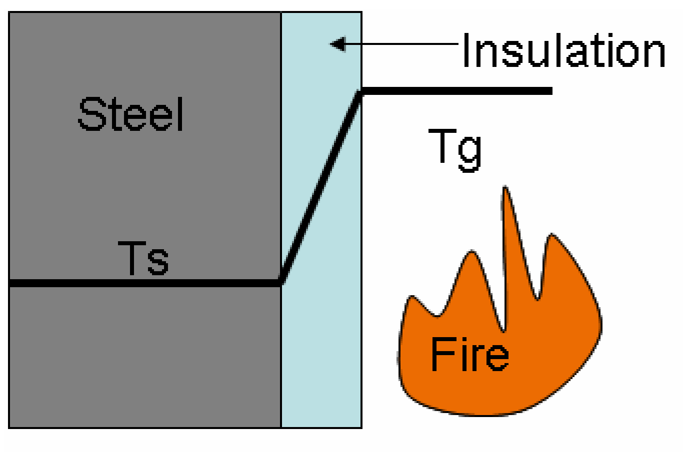

Figure 50 - Heat Transfer through Insulation

$$\Delta T_s = \frac{\frac{k_i}{t_i}}{\rho_s c_{ps}} \frac{A_i}{V_s} \left(\frac{1}{1+\xi} \right) \left(T_g - T_s \right) \Delta t - \frac{\Delta T_g}{1 + \frac{1}{\xi}}$$

Equation 11 - Change in Steel Temperature

Where zeta is a coefficient calculated using the equation in Equation 3 below.

$$\xi = \frac{\rho_i c_{pi} t_i}{2\rho_s c_{ps}} \frac{A_i}{V_s}$$

Equation 12 – Coefficient

The value of Ai/Vs was calculated using Equation 4 below.

$$\frac{A_i}{V_s} = \frac{(2d+3b_f-2t_w)}{A_s}$$

Equation 13 - Section Factor Equation

The time step factor Δt was calculated using Equation 5 below.

$$\Delta t \leq \frac{25000}{\frac{A_i}{V_s}}$$

Equation 14 - Time Step Factor

The other values supplemented into Equation 2 were taken from the tables below.

Material	Density kg/m³	Specific Heat J/kg	Thermal Cond. k_i W/m
Sprays			
Sprayed Mineral Fiber	300	1200	0.12
Perlite or Vermiculite	350	1200	0.12
Boards			
Gypsum Plaster	800	1700	0.20
Compressed Fiber Boards			
Mineral wool, fibre silicate	150	1200	0.20

Table 7 - Thermal Properties of Insulation Materials (Buchanan, 2001).

Recommended Specific Heat Capacity [J/kg °C]	Reference
$c_{ps} = 520$	ECCS Technical Committee 3 – Fire Safety of Steel Structures
$c_{ps} = 425 + 0.773T_s - 0.00169T_s^2 + 2.22x10^{-6}T_s^3$ $20\ °C \leq T_s \leq 600\ °C$	*Eurocode 3*
$c_{ps} = 666 + 13002/(738 - T_s)$ $600°C < T_s \leq 735°C$	
$c_{ps} = 545 + 17820/(T_s - 731)$ $735°C < T_s \leq 900°C$	
$c_{ps} = 650$ $900°C < T_s \leq 1200°C$	

Table 8 - Heat Capacities of Steel

For the analysis performed for each case, a spray-applied mineral fiber was used with a density of 300 kg/m^3 , a specific heat of 1200 J/kg°C, and a thermal conductivity of 0.12 W/m. The specific heat of steel was taken as 520 J/kg°C, and the density used was 7850 kg/m^3, which "remains essentially constant with temperature" (Buchanan, 2001). The calculated value for the change in steel temperature was then added to the previous temperature for steel to get the new steel temperature.

After the steel temperatures had been calculated for each fire condition, equations to obtain the yield strength and modulus of elasticity of the steel associated with those temperatures were used. The equations for these material properties are solely a function of temperature and do not account for any losses that may occur. They are taken from the *SFPE Handbook of Fire Protection Engineering* and can be seen below in Equations 6-9.

for $0 \le T \le 600C$

$$F_{yT} = \left(1.0 + \frac{T}{900 \ln\left(\frac{T}{1750}\right)}\right) F_{y0}$$

Equation 15 - Yield Strength for Temperatures between 0°C & 600°C

for $600C < T \le 1000C$

$$F_{yT} = \left(\frac{340 - 0.34T}{T - 240}\right) F_{y0}$$

Equation 16 - Yield Strength for Temperatures between 600°C & 1000°C

for $0 \le T \le 600C$

$$E_{T} = \left(1.0 + \frac{T}{2000 \ln\left(\frac{T}{1100}\right)}\right) E_{0}$$

Equation 17 – Modulus of Elasticity for Temperatures between 0°C & 600°C

for $600C < T \le 1000C$

$$E_{T} = \left(\frac{690 - 0.69T}{T - 53.5}\right) E_{0}$$

Equation 18 – Modulus of Elasticity for Temperatures between 600°C & 1000°C

The initial yield strength for these calculations, F_{y0}, was taken as 36 ksi, because when the cases being analyzed were built between the 1950's and 1970's; this was the predominant strength of steel used for construction. The initial modulus of elasticity, E_0

used was 29,000 ksi. The values calculated were then used as input for the RISA 2-D software analysis of the structural systems for the cases described below.

3 Case Studies

There were four cases researched and analyzed for this manual. Each building was constructed of structural steel, and each suffered from a major fire that lead to a partial or complete collapse. The four cases analyzed are McCormick Place, Chicago, Ill (1967), World Trade Center 5, New York, New York (2001), Alexis Nihon Plaza, Montreal, CAN (1986), and One New York Plaza, New York, New York (1970). The selected elements analyzed in each case are listed below in Table 3.

Case	Element Studied
World Trade Center 5	Floor Framing- Original Alternative Framing Systems Shear Tab Connections
Alexis Nihon Plaza	Welded Angle Connections Bolted Angle Connections
McCormick Place	Structural Steel Trusses
One New York Plaza	"Lessons Learned"

Table 9 - Case Elements Studied

3.1 McCormick Place

The first case illustrated in this manual is the fire that occurred at McCormick Place, Chicago, Ill in 1968. McCormick Place was a large exhibition hall that stood on the shore of Lake Michigan in Chicago, Ill. It was constructed in 1960 of reinforced concrete and structural steel and consisted of 3 levels of exhibition space, a large theater, restaurants, and a variety of other rooms and supporting spaces. Large steel trusses supported the roof of the structure; they spanned 210 feet column to column and cantilevered 80 feet on either side, as can be seen below in Figure 1.

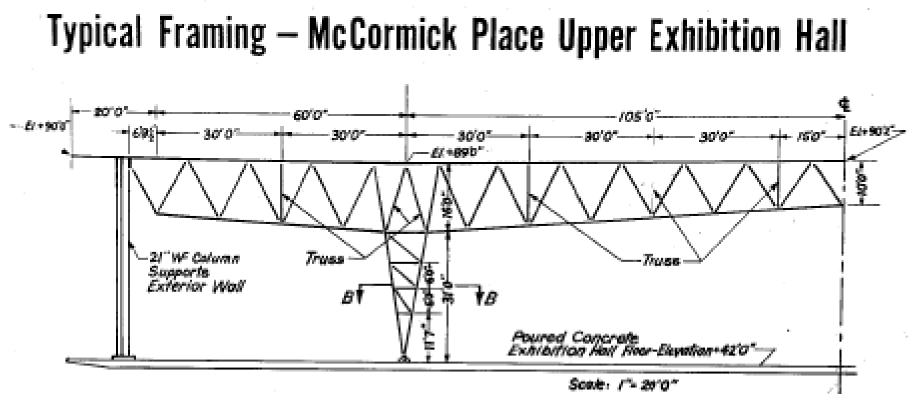

Figure 1 - McCormick Place Roof Truss (Jensen, 1967)

The trusses were 16' deep above the rigid frame support, 10 feet deep at the center, and 5'-3-1/2" deep at the cantilevered end. The truss and rigid frames that supported them were constructed of W14 wide flange members and were stabilized by smaller trusses running perpendicularly, similar to bridging. The column portion of these frames were protected from fire with a sprayed-applied fiber and encased "with metal lath and Gypsum vermiculite plaster," (Jensen, 1967) up to a height of 20 feet, but the trusses did not have any fire protection applied to them.

Once the fire broke out, it spread rapidly through the exhibition hall. Lack of fire protection caused excessive heating of the truss members, which then led to the collapse of the roof approximately an hour after the fire began (Jensen, 1967). It was stated that "the roof trusses started to buckle in the center, pulling the roof loose from the columns at the walls," (Juillerat, 1967).

This case was studied first and became the base case for the analyses performed for the others. The reason for this was that through study it clearly demonstrated the effectiveness of insulation. The calculations for this analysis were performed using the equations presented in Section 2.1.2. While this was the first step in the analysis of the other cases, it was less important because in those cases the steel had been protected with insulation prior to the fires.

3.1.1 Parametric Study of Spray-Applied Insulation

Analyses were performed in order to determine if spray-on insulation would have had any positive effect on the resistance of the steel truss to the fire. This was accomplished by comparing the performance of the unprotected steel truss to that of steel trusses with varying thickness of spray-applied insulation. The insulation was varied from ½" to 1- ½", and the results were then plotted on the same graphs to enable comparison of the effects of the insulation on steel temperature over time. Also depicted on the graph by a thick orange line is the critical temperature for steel, which is defined as "the temperature where steel has lost approximately 50 percent of its yield strength from that at room temperature," (Milke, 2002).

Steel	Temperature
Columns	538 °C (1,000 °F)
Beams	593 °C (1,100 °F)
Open Web Steel Joists	593 °C (1,100 °F)
Reinforcing Steel	593 °C (1,100 °F)
Prestressing Steel	426 °C (800 °F)

Figure 51 - Critical Temperatures for Various Types of Steel (Milke, 2002)

The yield strength of the steel (Fy) and the modulus of elasticity (E) of the steel during fire conditions were also investigated to illustrate the properties of steel under fire exposure.

The effects on the steel truss were modeled under the three fire conditions previously defined: the ASTM E-119 standard fire, a short duration-high intensity fire, and a long duration-lower intensity fire. These three fires each simulated a different scenario of fire intensity within McCormick Place and allowed for the analysis of the truss integrity under different scenarios.

3.1.2 Fire Exposure

The three graphs below illustrate the effect of insulation on steel temperature, yield strength and modulus of elasticity when exposed to the ASTM Standard E-119 fire.

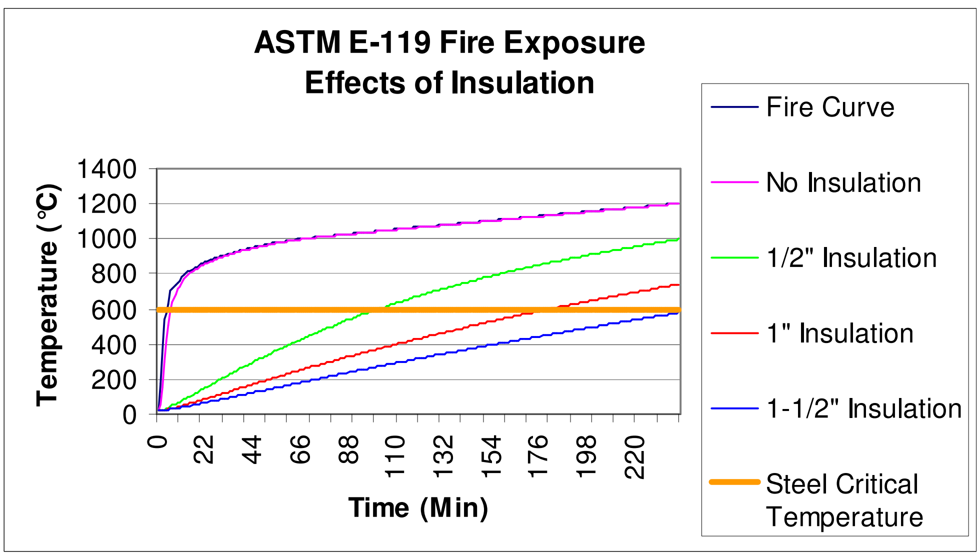

Figure 52 –Effect of Variable Insulation Thickness on Steel Temperatures

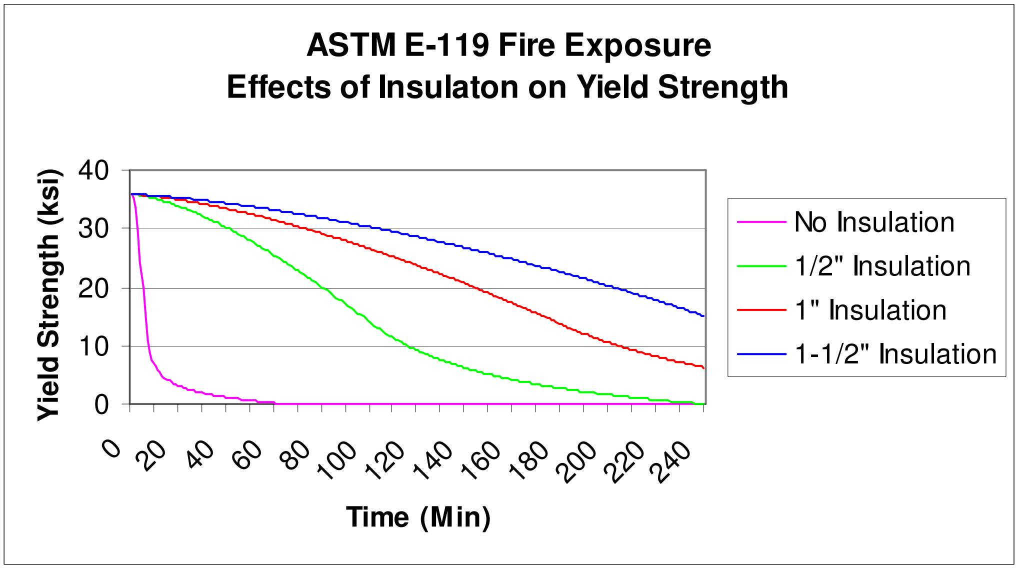

Figure 53 - Effects of Insulation Thickness on Yield Strength

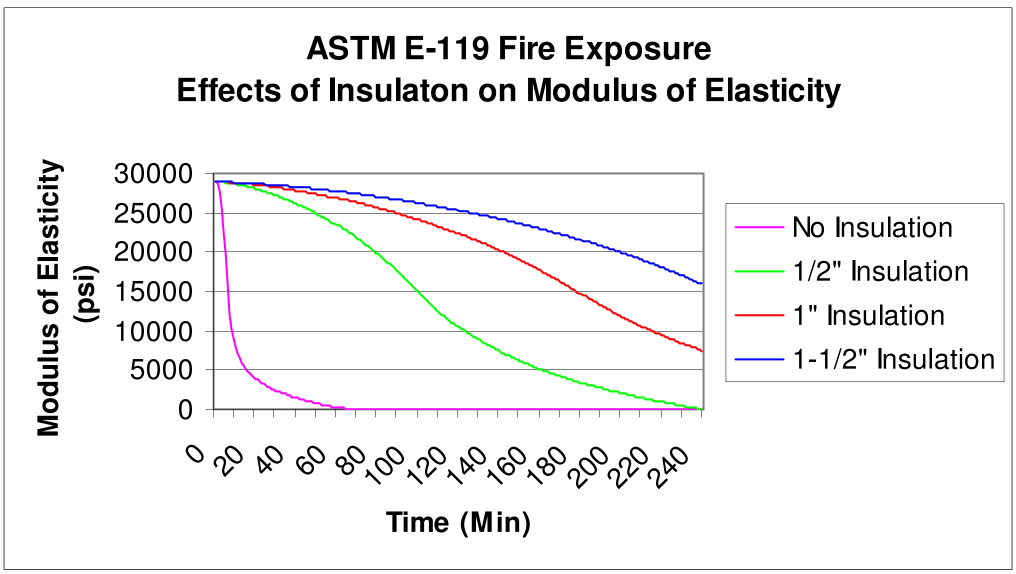

Figure 54 - Effects of Insulation Thickness on Modulus of Elasticity

The next three graphs show the effect of insulation on steel temperature, yield strength and modulus of elasticity when exposed to a short duration-high intensity fire.

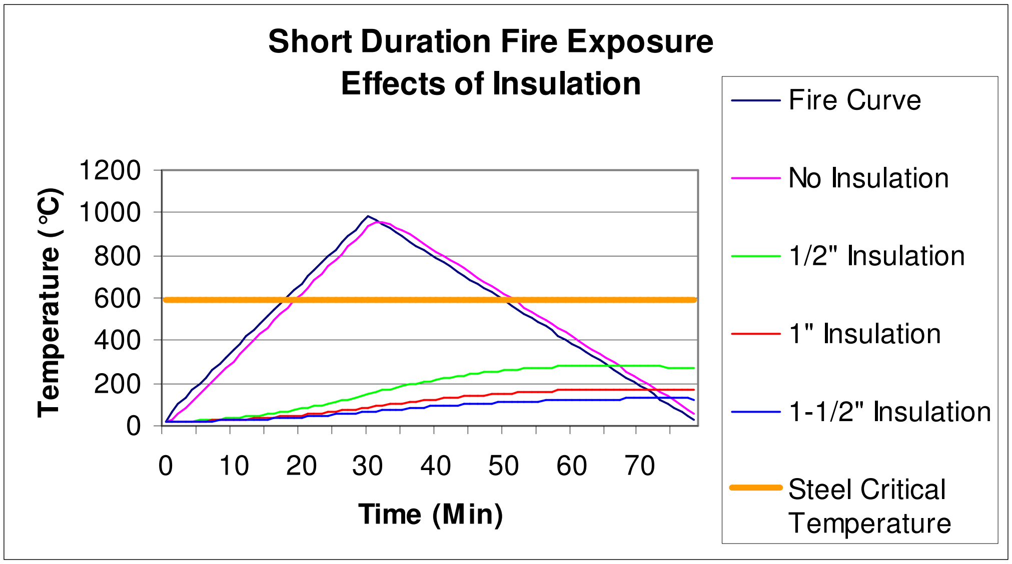

Figure 55 - Effect of Variable Insulation Thickness on Steel Temperatures

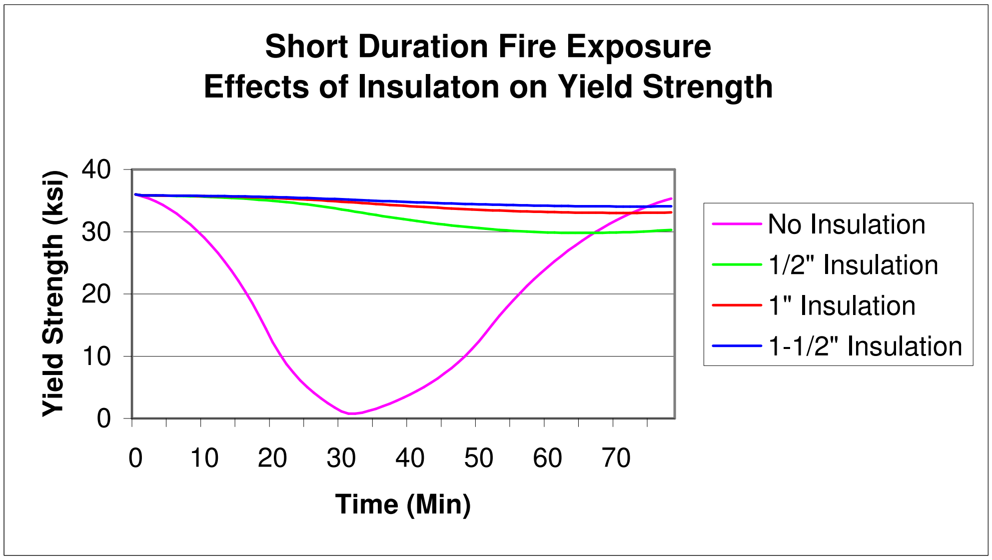

Figure 56 - Effects of Insulation on Yield Strength

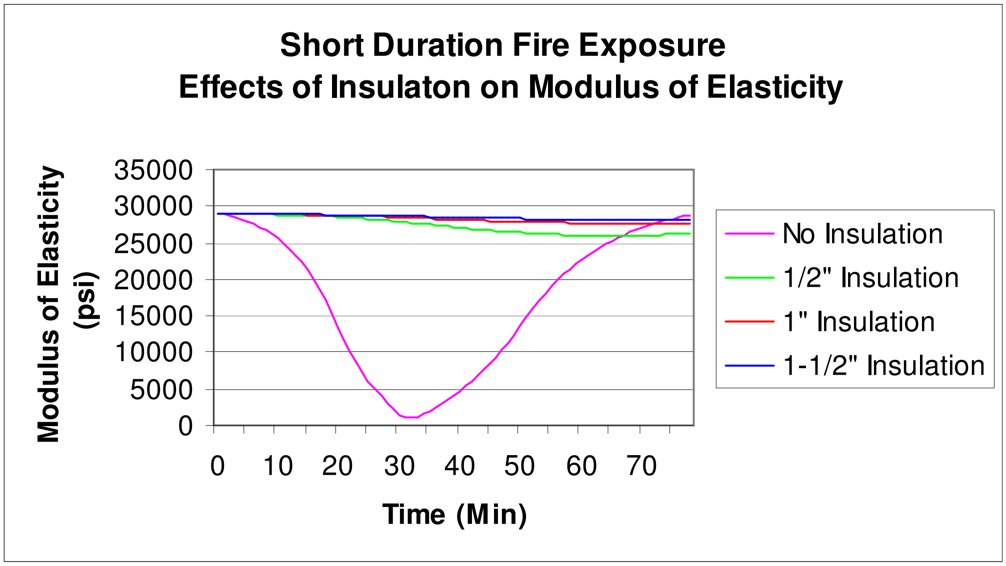

Figure 57 - Effects of Insulation on Modulus of Elasticity

The last three graphs show the effects of insulation on steel temperature, yield strength, and modulus of elasticity when exposed to a long duration-lower intensity fire.

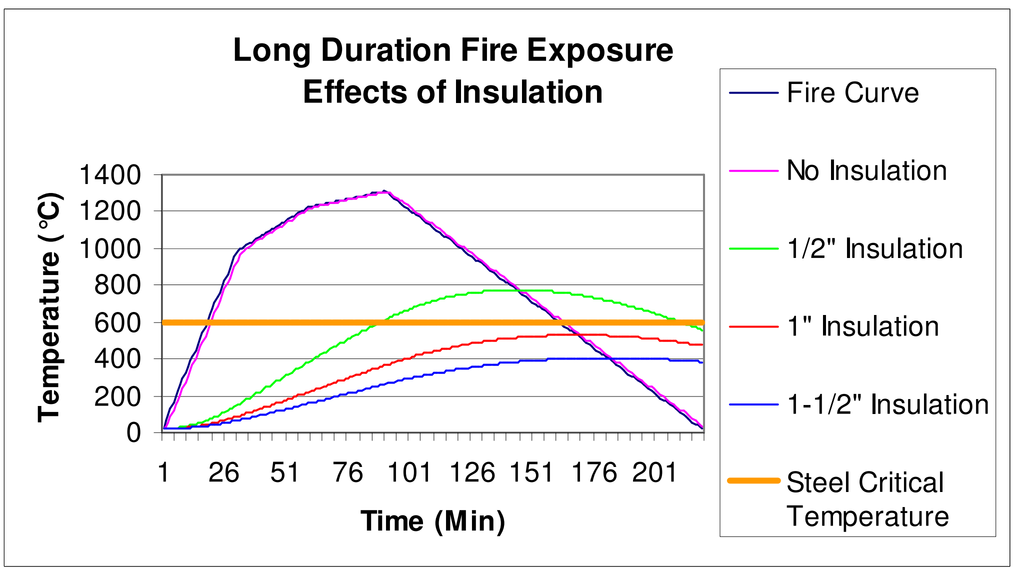

Figure 58 - Effect of Variable Insulation Thickness on Steel Temperatures

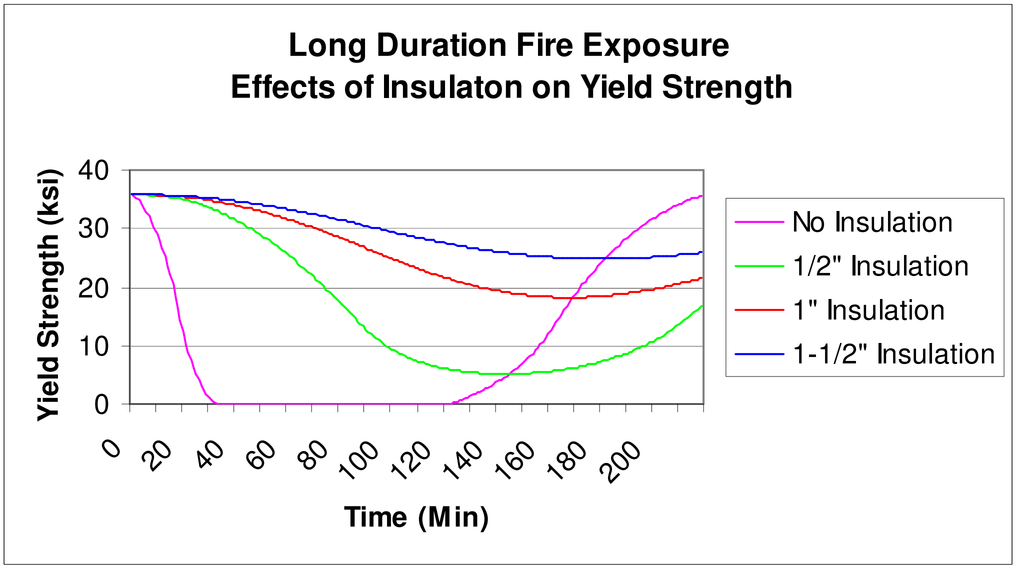

Figure 59 - Effects of Insulation on Yield Strength

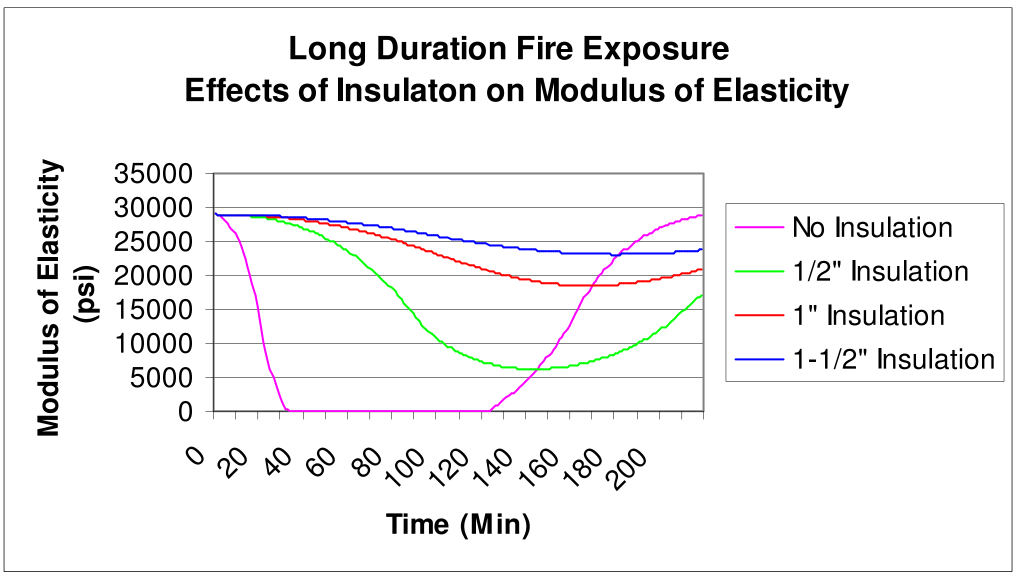

Figure 60 - Effects of Insulation on Modulus of Elasticity

As seen in the graphs, Figures 6, 9, and 12 above the use of any thickness of spray-on insulation would have reduced the temperatures of the steel truss significantly during all three design fires. In each case the greatest reduction of heat occurs early in the fire which is important because it may have given the fire department sufficient time to get the fire under control before excessive heating and structural instability occurred.

The yield strength (Fy) and modulus of elasticity (E) of the steel truss are important factors, because when these values are reduced, the strength of the steel is reduced, which could lead to failure. In all three cases, after one hour (approximate time of failure) with ½" insulation the loss of yield strength would have been 10 ksi as compared with the truss with no insulation that would have lost approximately all of its strength. Similarly, for the moduli of elasticity graphs, the loss for the ASTM E-119 and long duration fires is 5000 psi after one hour. For the short duration fire the loss is about 4000 psi for ½" insulation thickness. The loss for the cases with more insulation is less than that for the ½" insulation thickness. Under the short and long duration fire exposures one can see that the truss without insulation regains strength as the fire decays. This is true to a certain extent but unlike the graph depiction, it will never regain its full capacity. The graphs are off because the equations used to calculate these values (Equations 6-9) only account for temperature and no other factors.

One difference to note between the short duration fire and the ASTM E-119 and long duration fires is the performance of the insulation thickness. Under the short duration fire exposure, there is little difference in the performance if insulation is provided. This is due to the fire peaking early and gradually cooling off. The short fire timeline enabled all of the insulation thickness to perform at approximately the same levels. Unlike the short duration fire, a greater thickness is more effective in the ASTM E-119 and long duration fires. The differences between 1" and 1½" of insulation are not as great as the difference between the ½" and 1" of insulation. Because of this, it would be a good decision to have at least 1" of insulation under these fire conditions.

The analysis performed in this investigation does not determine the mode of failure, however, it was discussed in the investigative report from observation that the trusses buckled in the center and a collapse of the roof ensued. This is the most probable situation since the compression members (bottom chord of the truss) would have heated the quickest and experienced warping and twisting due to the extreme temperatures produced by the fire. The center of the truss also did not have any diagonal members to support the top and bottom chord and therefore had the least resistance to failure as compared to the other pieces of the truss.

3.2 World Trade Center 5

World Trade Center 5 was a nine-story office building that was completed around 1970. The building's dimensions were 330 ft. by 420 ft. with 30 ft. by 30 ft. bays. The roof and ninth floor were constructed using conventional steel framing; the eighth floor and those below were constructed using column trees, as can be seen below in Figures 15 and 16. These trees were constructed of a 4-ft.-W24x61 stub girder shop welded to the columns. In the field W18x50 floor girders were connected to the stub girders using shear tab connections. The W24x61 member is a Canadian shape that is similar to a W24x62 with a thinner web and shorter depth. The floor system was constructed of 4" lightweight concrete on 1-1/2" metal deck, and was attached to the floor beams and girders using shear connectors to create a composite floor system. All of the structural members were fire-protected with a sprayed-on mineral fiber that provided a 2-hour fire rating to the floors and a 3-hour rating to the columns (Barnett, 2002).

Fire in this building broke out due to debris that impacted the building from the collapse of the World Trade Center towers on September 11, 2001. The subsequent fires caused shear connectors of the column tree assemblies to fail initiating collapse from the eighth floor through the fifth floor.

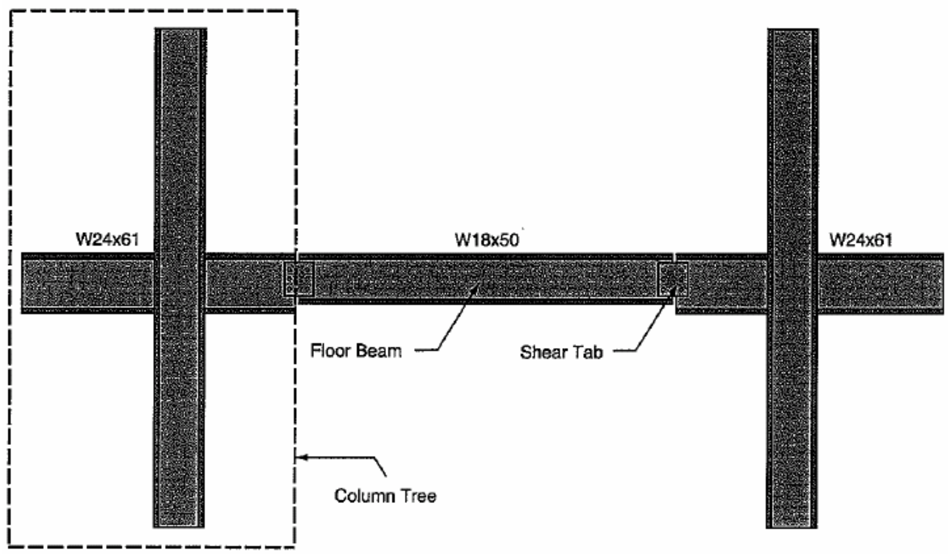

Figure 61 - Typical Column Tree System (Barnett, 2002)

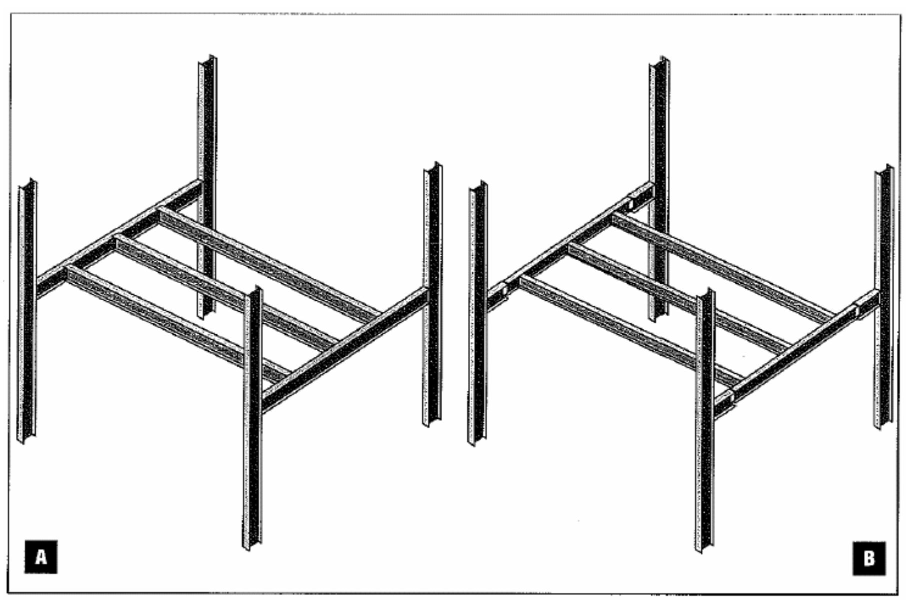

Figure 4-3 Typical interior bay framing in WTC 5. (A) Floor 9 and roof level. (B) Floors 4, 5, 6, 7, and 8.

Figure 62 - Interior Bay Framing of WTC 5 (Barnett, 2002)

From the investigative report of this failure, it was determined that the shear tab connections failed due to secondary tensile forces developed from catenary action. However, in order to come to this conclusion and to provide a thorough understanding of the failure, a much more in depth analysis was performed. The analysis began with investigating the drop-span section of the column tree assembly for bending capacity and deflection as a function of time and temperature (calculated using the equations presented in Section 2.1.2). This same analysis was then done for the two alternative situations of continuous W24x61 and W18x50 girders replacing the column tree assembly.

The next step in the analysis was the investigation of the shear tab connections. These calculations followed those performed in *Appendix B: Structural Steel and Steel Connections* (Fisher, 2002) of the *World Trade Center Building Performance Study* (FEMA). These calculations were extended to analyze the bolted connections from 20°C (room temperature) to 650°C depending on the fire exposure used.

3.2.1 Floor Framing Analysis

Performance of the floor system under fire condition was modeled using the RISA-2D Educational software. In order to describe fire conditions for input into RISA-2D, time-step temperature calculations were done to determine the temperature of the steel with ½" insulation, 1" insulation, and 1-1/2" spray-applied mineral fiber insulation when exposed to the ASTM E-119 fire, a short duration-high intensity fire, as well as a long duration-lower intensity fire. Once the temperatures of the steel were calculated (at every 50° C, gas temperature), the yield strength and modulus of elasticity's that corresponded to those temperatures were calculated using Equations 6 - 9 from the *SFPE*

Handbook of Fire Protection Engineering. The moduli of elasticity's were then input into RISA to calculate the deflections associated with the increase in the temperature of the steel due to fire exposure. In order to approximate the behavior of the actual composite floor system, the moment of inertia for the drop-span girder (W18x50) was adjusted to be equal to the moment of inertia for the composite system. Also, the distributed load input to the program only reflected the live load to which the system would have been subjected, in order to account for shoring during construction, or any original camber that may have been present in the girder.

The original model was then revised to replace the column tree system with (1) a W24x61 continuous girder and (2) a W18x50 continuous girder. The moments of inertia for both these beams was adjusted to reflect the moments of inertia of the composite systems, and both of these framing systems were exposed to the same loads seen by the original model. Both of these alternatives also used adjusted moduli of elasticity's to reflect temperature for the varying insulations, as done in the original analysis.

The deflections for each of these systems for exposure to the three different fire exposures previously mentioned can be seen in the figures 17, 18, and 19 below for ½" insulation thickness.

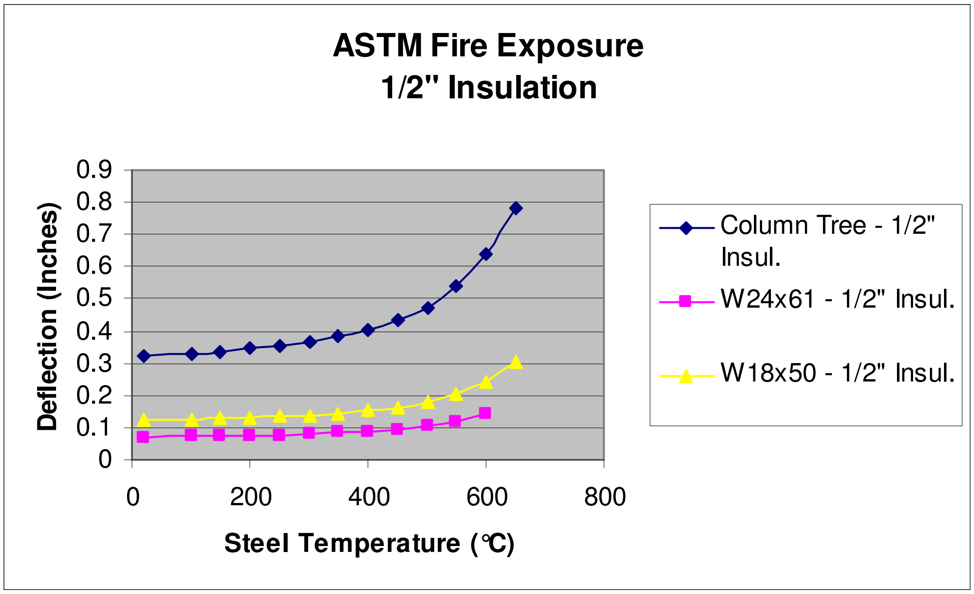

Figure 63 - Deflections under ASTM E-119 Fire Exposure

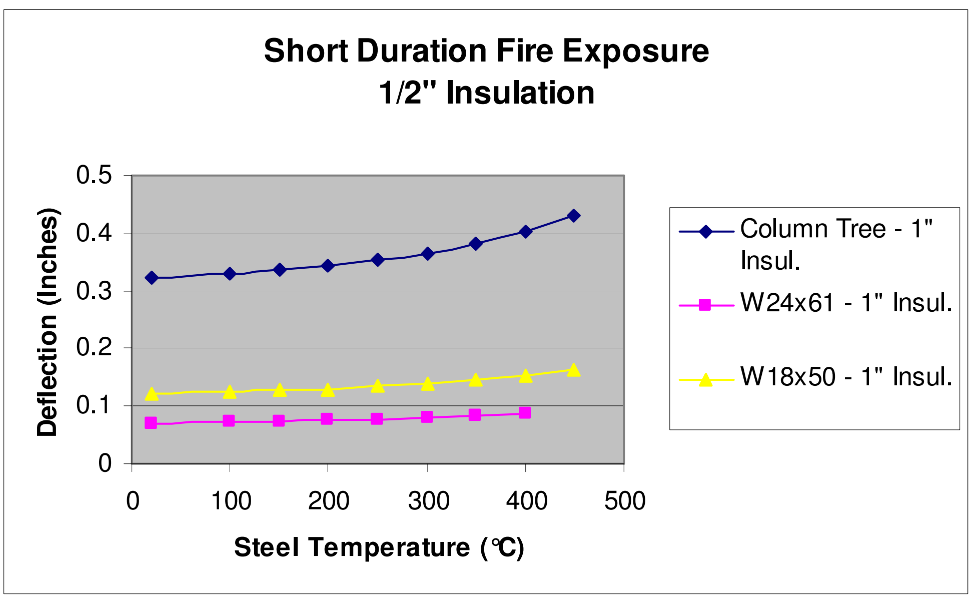

Figure 64 - Deflections under Short Duration-High Intensity Fire Exposure

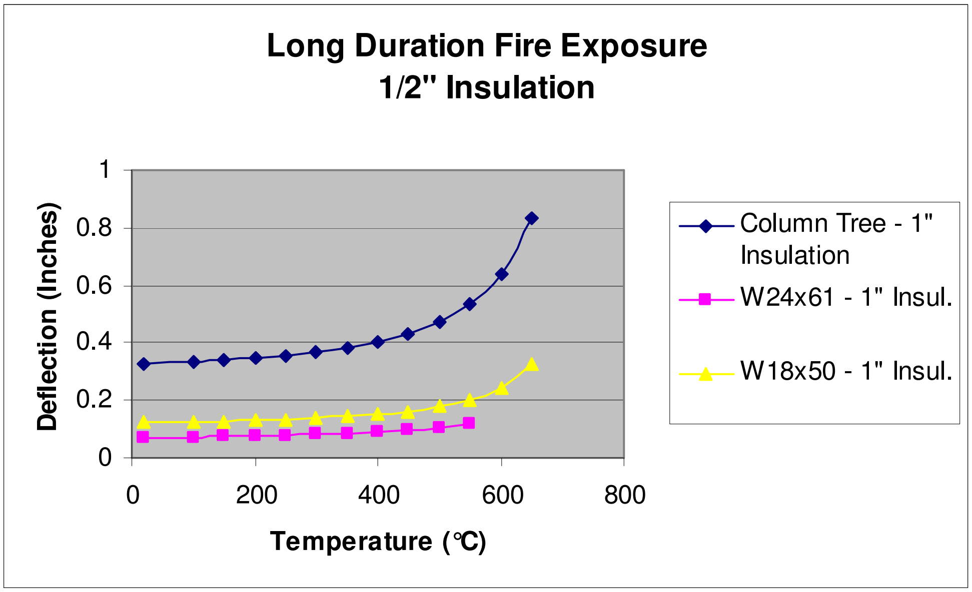

Figure 65 - Deflections under Long Duration-Lower Intensity Fire Exposure

The next three graphs show the deflections for each design fire using 1" insulation thickness.

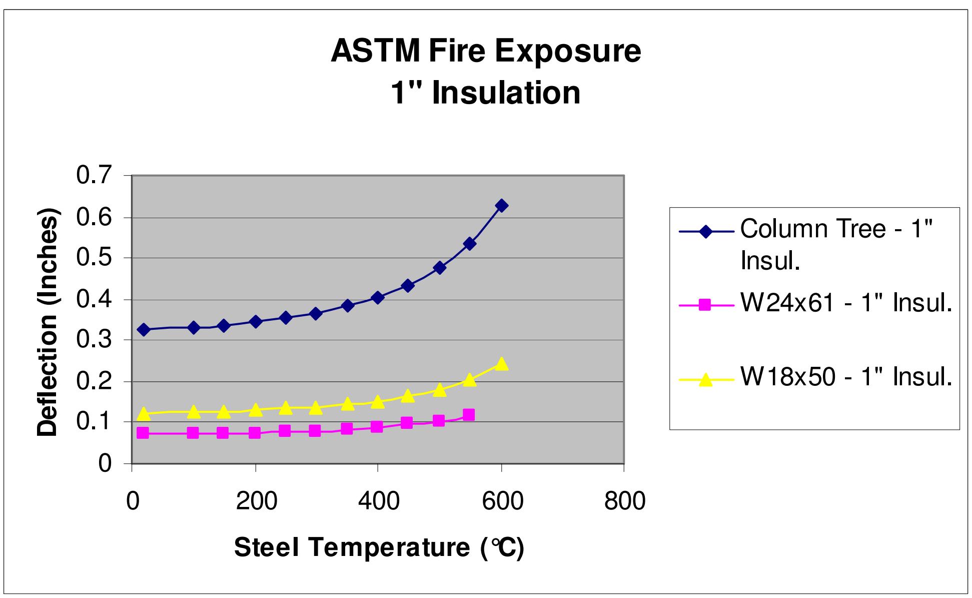

Figure 66 - Deflections under ASTM E-119 Fire Exposure

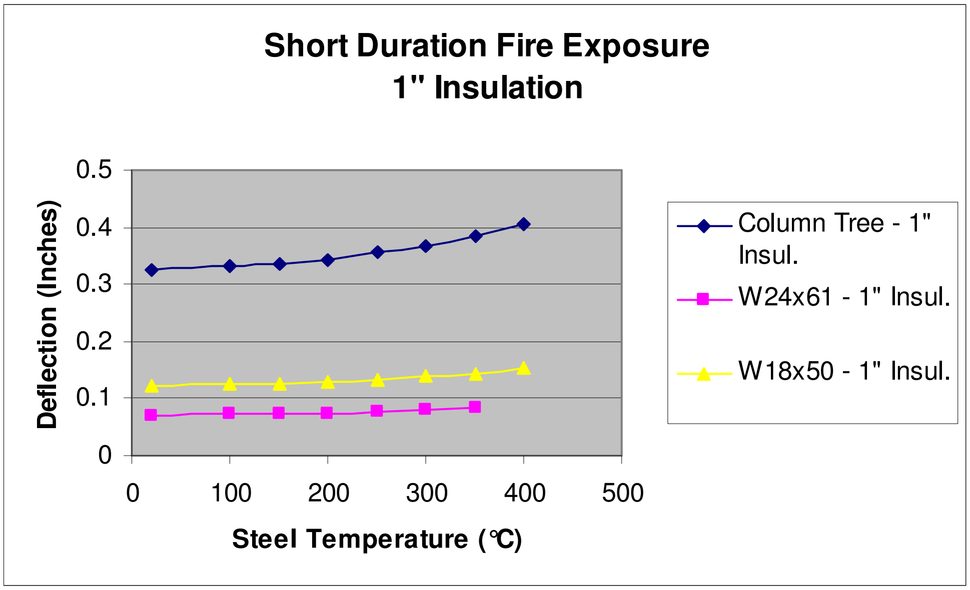

Figure 67 - Deflections under Short Duration-High Intensity Fire Exposure

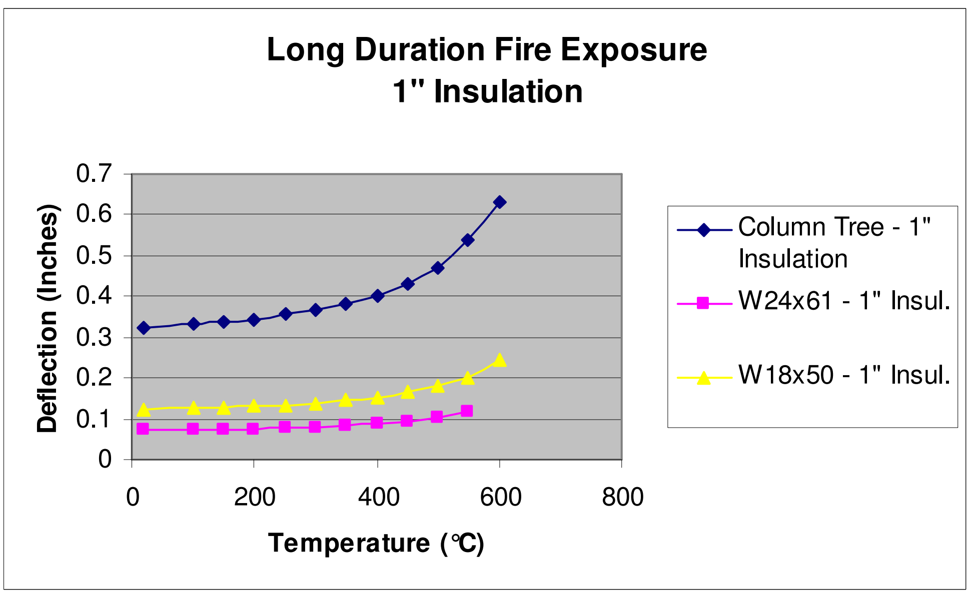

Figure 68 - Deflections under Long Duration-Lower Intensity Fire Exposure

The next three graphs show the deflection for the original and alternative framing systems with 1-1/2" insulation thickness, subjected to the three design fires.

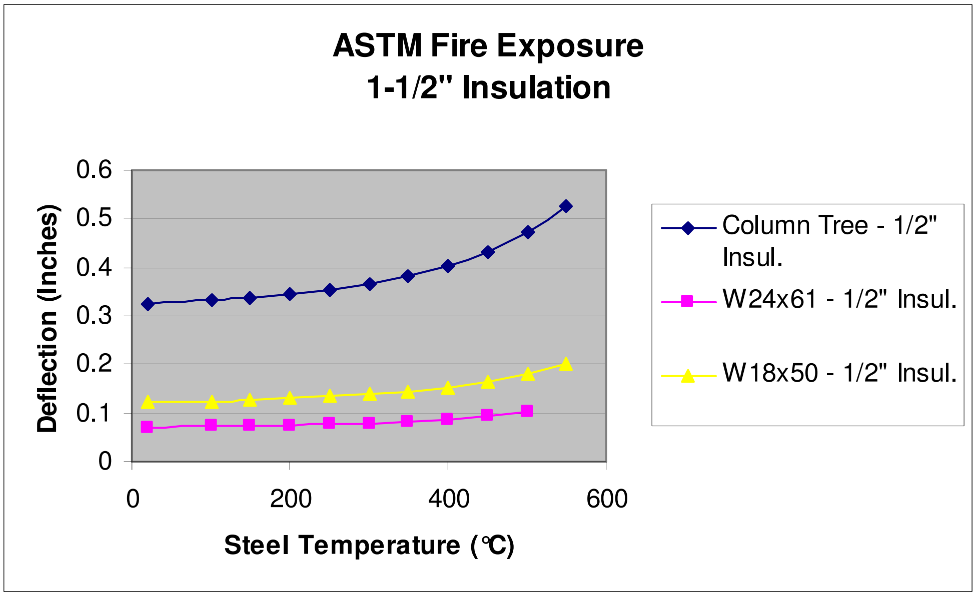

Figure 69 - Deflections under ASTM E-119 Fire Exposure

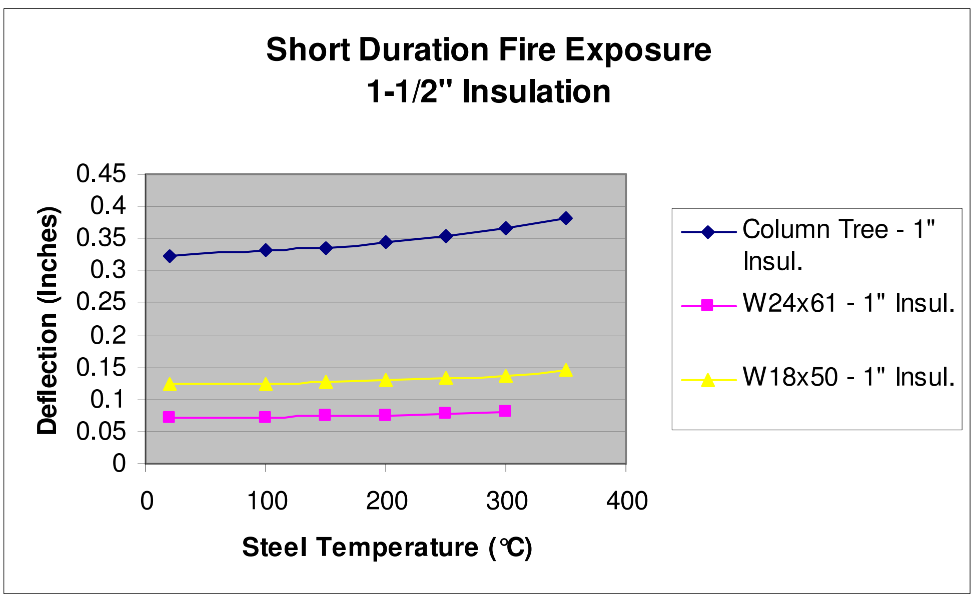

Figure 70 - Deflections under Short Duration-High Intensity Fire Exposure

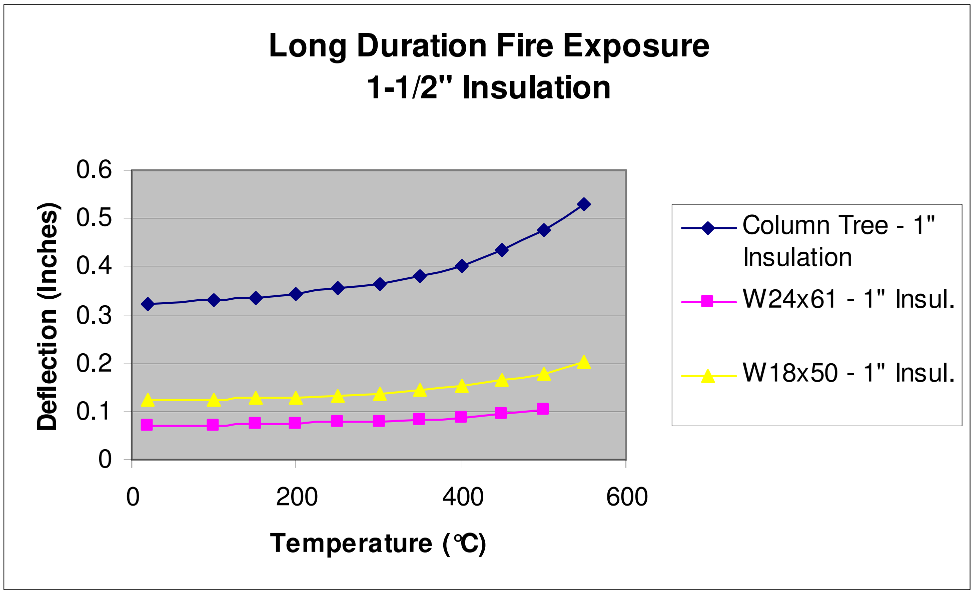

Figure 71 - Deflections under Long Duration-Lower Intensity Fire Exposure

For all three insulation thicknesses, one can see that the behavior under the ASTM E-119 curve and the long duration fire are very similar. It can also be seen that as the insulation thickness becomes greater, the deflection drops. An interesting point to note from these graphs is the similarity in behavior of the deflection patters, but the large difference in the amount of deflection between the continuous beams and the column tree assembly.

One should also note that the deflection limits of 0.73" (L/360) in the original case (L=22') and 1" (L/360) for the two continuous span alternative cases (L=30') is never exceeded under all three-fire conditions. This shows that all three framing systems could have theoretically handled the loads under the exposed fire temperatures so the system would not have failed due to deflection.

The technical report on this failure states "the structural damage due to the fires closely resembled that commonly observed in test assemblies exposed to the ASTM E119 Standard Fire Test," (Barnett, 2002). It then goes on to say that the "local collapse appeared to have begun at the field connection where beams were connected to shop-fabricated beam stubs and column assemblies," (Barnett, 2002).

3.2.2 Shear Tab Connection Analysis

The next step in the analysis process was to examine the shear tab connections where the failure appeared to have begun. In *Appendix B of the World Trade Center Building Performance Study* (Fisher, 2002), the three-bolt capacity, double shear capacity, and the tensile capacity of the shear tab connections were calculated at room temperature and at 550° Celsius. This was done by adjusting the yield strength values of the bolts by a factor determined from Figure 26 below; extracted from *Appendix A, Overview of Fire Protection in Buildings, WTC Building Performance Study.*

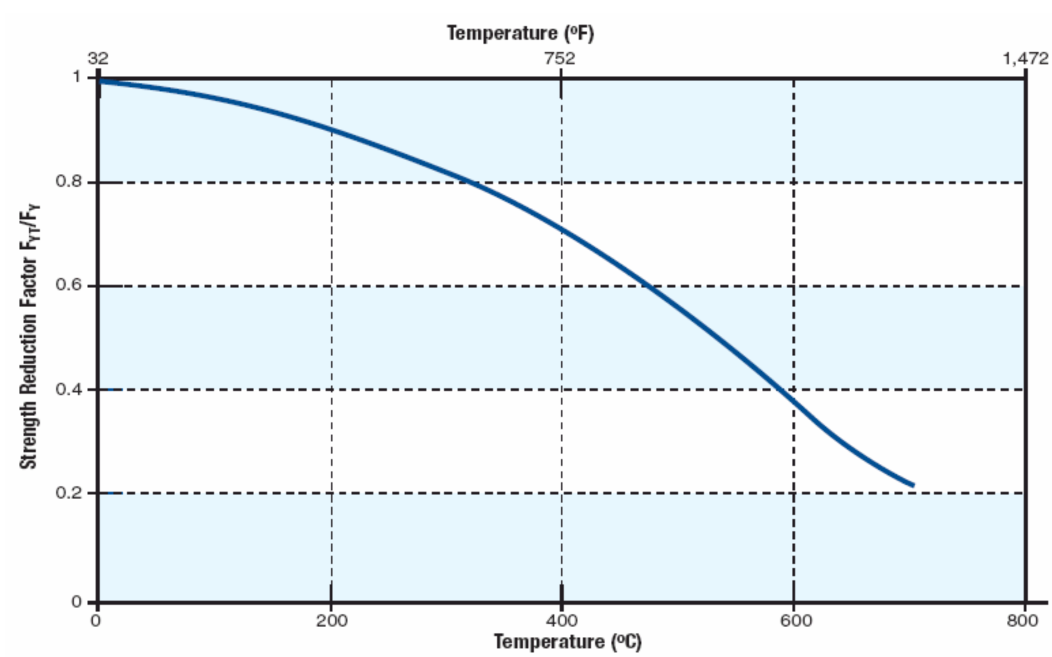

Figure 72 - Strength of Steel at Elevated Temperatures (qt. Milke, 2002)

For the purpose of this project, these calculations were extended to obtain the capacities of the shear tabs in the temperature range from 20°C (room temperature) to 700 °C. The critical temperature for steel beams/girders is listed in Figure 5 above as 593°C and it is a theoretical assumption that connections will degrade at the same rate or slower as the members they are connecting. If this is true then the connections should not have failed until about 600°C, and from the technical report it states failure at a temperature of 550°C or less.

Shown below, in Table 4, are the calculated values for the three-bolt capacity of the connections, the 3-bolt shear capacity, double shear capacity, the tensile capacity, and the block shear capacity. While block shear was not a design requirement defined in the *1963 AISC Specification for the Design, Fabrication & Erection of Structural Steel Buildings,* it has been calculated for research purposes. The tabulated values for the double shear capacity are compared to the plastic shear capacity of the W24x61 girder and W18x50 girder and are greater; therefore the connections could handle the shear produced under the fire conditions. These shear values do not include the added shear that was produced from the collapsed floor weights. From Table 4 it can be seen that the governing mode of failure was from larger tension forces produced by catenary action than could not be withstood by the shear tab connection. Since these forces are not present under normal conditions, they would not have been originally designed for. The second mode of failure seen from the table would have been block shear, which was also not a design requirement in the *1963 AISC Specification for the Design, Fabrication & Erection of Structural Steel Buildings*. The capacity of the block shear at 200°C is less than the double shear capacity of the bolts and W24x61 girder at 550°C. In order to improve this, the thickness of the plate would have to double or a different bolt configuration could be tried.

		W24x61	W18x50	Shear	Shear	Tensile	Block Shear
Temperature	Strength Factor Ratio	Vp	Vp	Vu	Ru (D)	Ru	ϕRn
0	1	231.29	153.93	151.88	232.72	90.72	121.64
20	.99	220.98	153.36	150.36	230.39	89.81	120.42
100	.95	219.72	145.69	144.28	221.08	86.18	115.55
150	.93	215.10	142.62	141.24	216.43	84.37	113.12
200	.90	208.16	138.02	136.69	209.45	81.65	109.47
250	.85	196.59	130.36	129.09	197.81	77.11	103.39
300	.82	189.66	125.76	124.54	190.83	74.39	99.74
350	.77	178.09	118.09	116.94	179.79	69.85	93.66
400	.70	161.34	107.35	106.31	162.90	63.50	85.14
450	.65	150.35	99.68	98.72	151.27	50.97	79.06
500	.57	131.83	87.42	86.57	132.65	51.71	69.33
550	.48	111.02	73.61	72.90	111.71	43.55	58.38
600	.39	90.20	59.81	59.23	90.76	35.38	47.44
650	.29	67.07	44.47	44.04	67.49	26.31	35.27
700	.21	40.57	32.21	31.89	48.87	19.05	25.54

Table 10 - Steel Connection Capacities

3.2.3 Discussion

Catenary action is a phenomenon that occurs in composite beams as a result of thermal expansion in the member due to elevated temperatures. In all buildings, beams and girders have a certain amount of end restraint, even if simply supported. "The end restraints, although negligible at normal temperature, become significant at elevated temperature because of restraint to thermal expansion may cause enormous internal force and moment within the structural member," (Yu, 2005).

Catenary action was studied for the first time after the Cardington fire tests, conducted in 1995 on an 8-story composite steel frame. From these tests it was determined that when fire temperatures were below 400°C the slab acted as an extension of the compression flange of the structural steel and had little influence on the system, but when temperatures exceeded 500°C the slabs became a very influential part of the system. "The influence of membrane tensions in the slab cannot be ignored, particularly when a fire compartment is subject to high horizontal restraint from surrounding cool, stiff structure, or when it is vertically supported around its perimeter at protected lines of support. When the double-curvature deflections of floor slabs become large the influence of tensile membrane action can become very important in supporting the slab loading," (Huang, 2002). This tensile membrane action or catenary action can be useful in preventing a progressive collapse; however if the tensile forces developed are larger than the tensile capacities of the connections, the system will fail. This is the mode of failure that was believed to have occurred in the World Trade Center 5 column tree assemblies (Barnett, 2002). In Figure 34 below, one can see the catenary action developed in the upper floors that had typical structural framing.

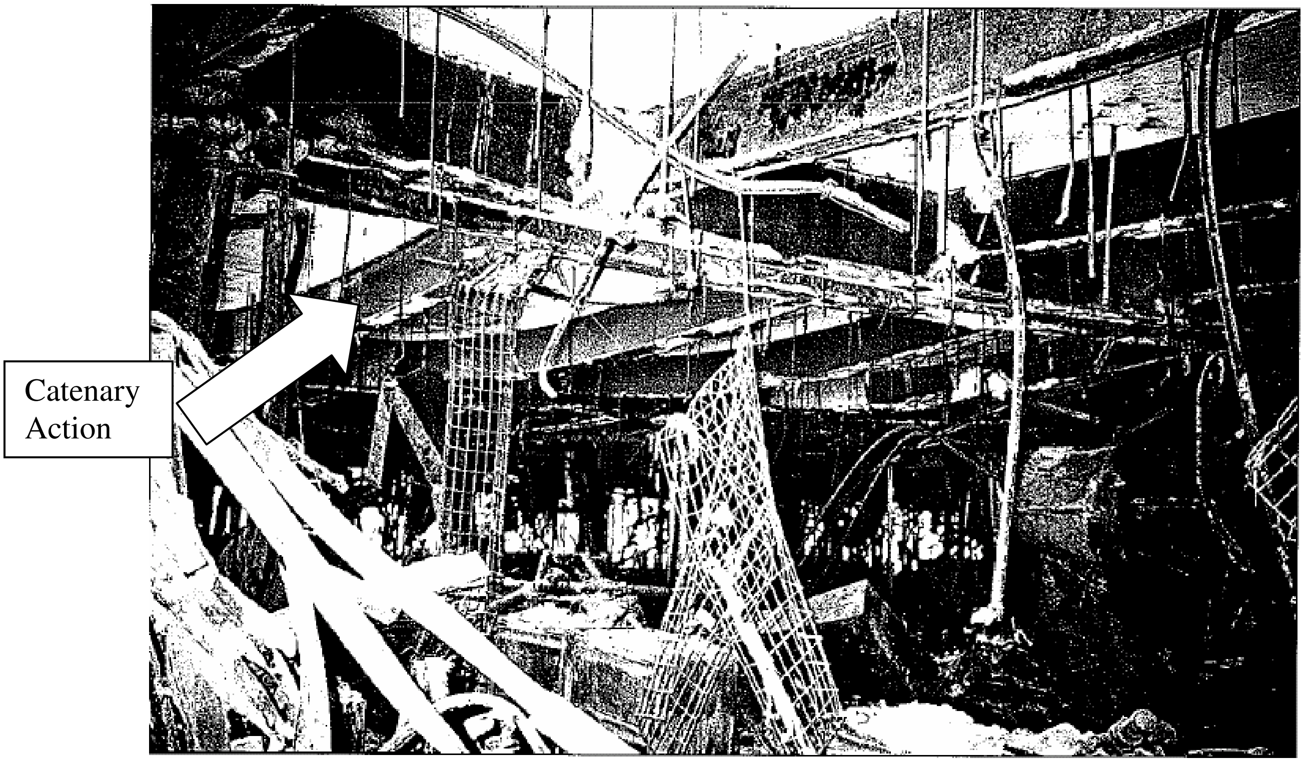

Figure 73 - Catenary Action in WTC 5

Since the destruction of the World Trade Center Plaza, there has been much research done to investigate this phenomenon. Currently, "tensile catenary action of floor framing members and their connections has been neither a design requirement nor a design consideration for most buildings," (Barnett, 2002). One paper entitled "Considering Catenary Action in Designing End-restrained Steel Beams in Fire" provides simplified equations to calculate these forces as a function of temperature for end restrained beams. Calculations using these equations were then compared to calculations done using the traditional design method to determine any advantages to using this approach. It was found that there was an advantage to using this new method when comparing beams with only axially restrained ends, but there was little advantage when the members had a fair amount of rotational end restraint (Huang, 2005).

One way to advance this study would be to gain a greater understanding of the catenary forces that were created, and compare those values with the tensile strengths of the plates. Another way would be to alter the RISA model to incorporate column degradation as a function of time and temperature to get a more realistic model.

3.3 Alexis Nihon Plaza

The ten-story office building (referred to as the 15-story office building in the official report) that was part of the Alexis Nihon Plaza was built atop a five-story shopping/parking facility sometime after 1950. On the evening of October 26, 1986 at around 5 p.m., a fire broke out on the 10th floor and spread through stairwell B (see Figure 28) up to the 16th floor (there was no 13th floor). At approximately 10:30 p.m., a 30 ft by 40 ft. section of the 11th floor, near where the fire was believed to have originated, collapsed onto the 10th floor (Isner, 1986). This was the only section of the building that suffered collapse, and upon investigation, the collapsed members did not exhibit the properties commonly found of members exposed to high temperatures, “such as bending, elongation, or twisting,” (Isner, 1986). The collapse is believed to have occurred because the welds connecting the girders and columns failed causing the collapse of a 30 ft by 40 ft. section of the 11th floor.

The building was constructed of a structural steel frame of 11 – 30 ft. by 30 ft. bays and 3 – 15 ft. by 30 ft. bays, as can be seen in Section 3.3, Figure 28. The girders (W24x76’s), beams (W18x40’s), columns and metal decking were all fire-protected with sprayed-on mineral fiber providing a 2 – 1/2 to 3 hour fire rating for the members. “The girder – to – column, beam – to – column, and beam – to – girder connections were made using double clip angles,” which were “welded to the beams and bolted to the columns or girders,” (Isner, 1986).

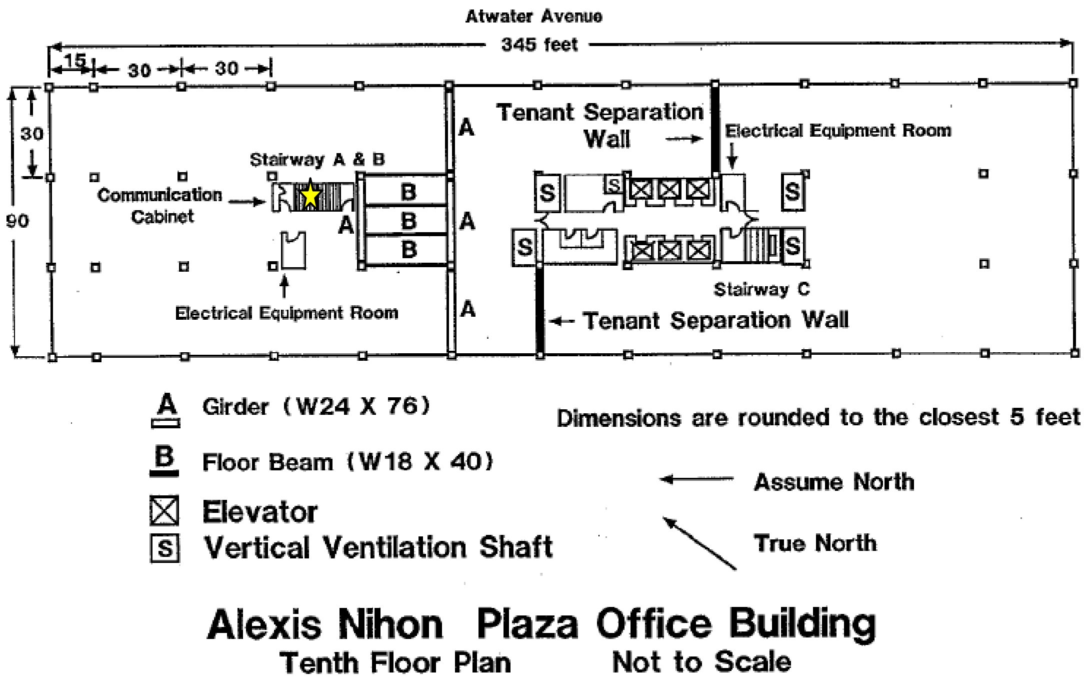

Figure 74 - Alexis Nihon Tenth Floor Layout

3.3.1 Framing Analysis

To begin the analysis of this system, the girders material properties were calculated using the equations in Section 2.1.2. Using these values, RISA-2D WebDemo, was utilized to determine the end shear and moments in the girder. These numbers were then used for analysis of the weld shear capacity for a variety of angle sizes, weld thicknesses and strength. This analysis was also done for the bolts. Comparison of these values along with the capacity of the girder, led to the determination of the failure mode at various temperatures for various sizes of angles. These modes of failure are summarized and discussed below.

3.3.2 Angle Connections

The information on the connections for this case was extracted from *High-Rise Office Building Fire, Alexis Nihon Plaza*, which was the investigative report of the failure. The report did not provide any data on the size of the angles, weld thickness and strength, or the number of bolts for each connection. Data on the connections could not be found in a wider search of the literature. To get a range of data for analysis, angles of various sizes were used to calculate the shear capacity of the welds and the tensile, shear, double shear, bearing, and block shear capacities of the bolts. The capacities are calculated based on geometry and material properties; they would be compared with the forces predicted by analyses. Weld capacity is calculated using the moment produced from the girder, and the length of weld, which changes with the size of the clip angle. The shear capacities of the welds, bolts, and girder can be seen in the graphs below. The bolts were calculated for an L8x8x1/2" angle and an L8x8x1/4" angle. The shear capacities of the welds were calculated for weld metal with both a 60 ksi and 70 ksi tensile strength. The steel temperatures were calculated for ½", ¾", 1", and 1-1/2" insulation thickness for the three fire exposures. The 3/4" insulation thickness is the thickness that was present in the original design. To illustrate the difference in shear capacities of the structural elements, the capacities of the welds, bolts and girder were placed on a single graph for each of the three design fires. The solid red line seen in these graphs is the end shear produced by the girder on the connections. The three graphs shown below in Figures 35-37 are for a ½" angles and a 7/16" fillet weld with a 60 ksi electrode. The 60-ksi weld metal was chosen because it produces the worst-case scenario.

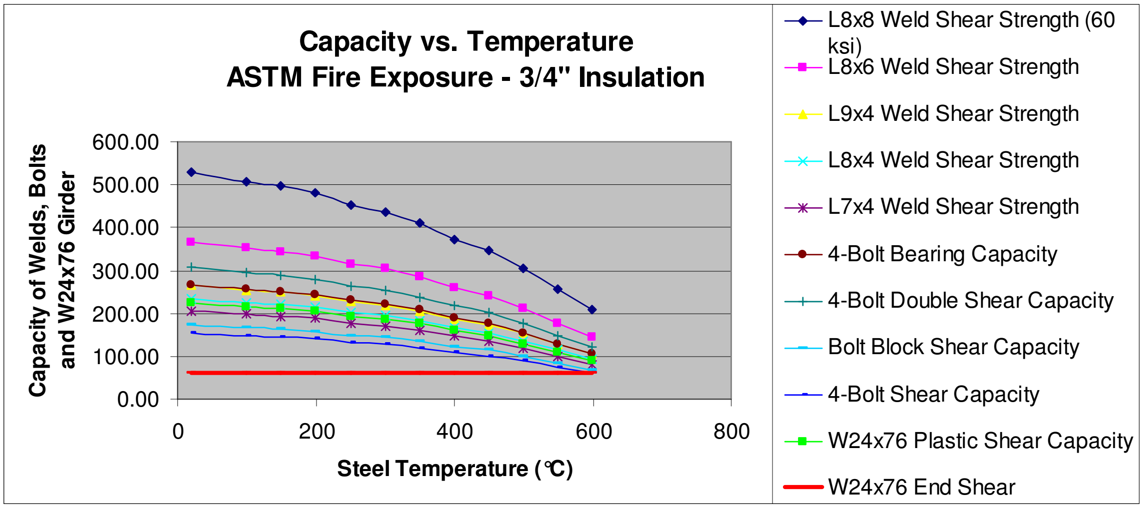

Figure 75 - Shear Capacities - L8x8x1/2" Angle & 7/16" fillet - 60 ksi weld metal

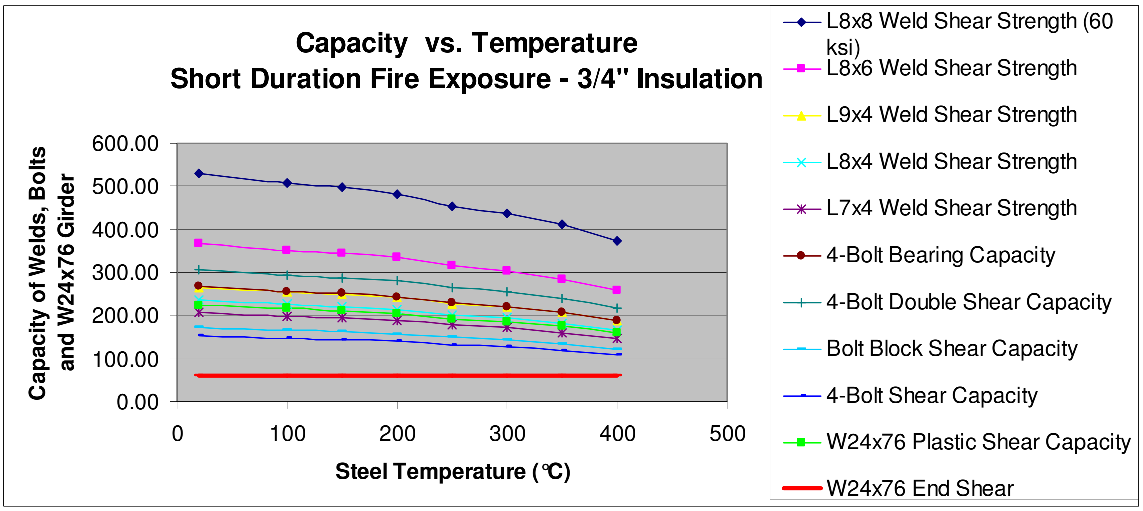

Figure 76 - Shear Capacities - L8x8x1/2" Angle & 7/16" fillet - 60 ksi weld metal

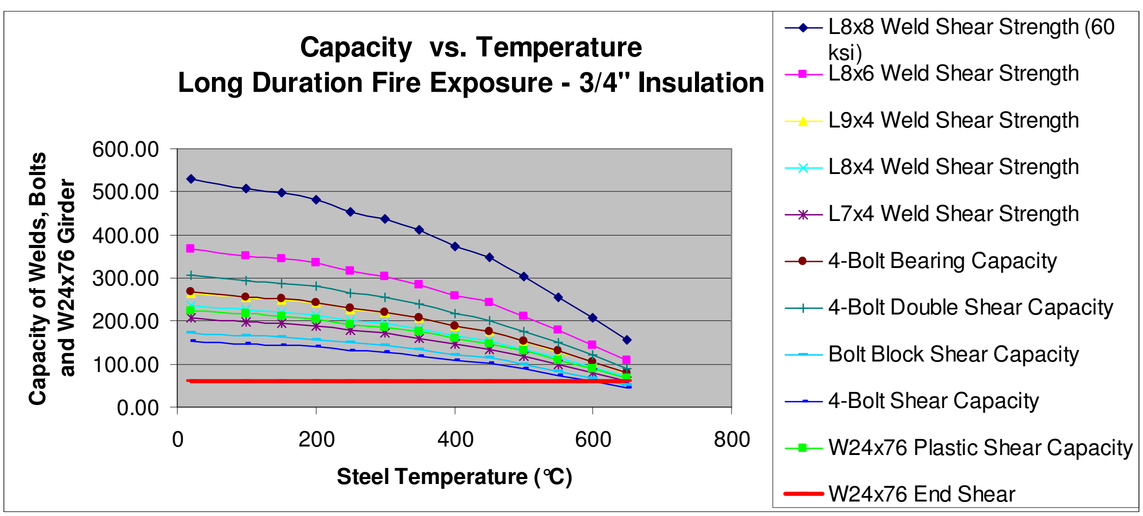

Figure 77 - Shear Capacities - L8x8x1/2" Angle & 7/16" fillet - 60 ksi weld metal

The three graphs shown below, Figures 38-40, illustrate the shear capacities for the elements listed above, for a variably sized ½" angles and a 1/4"-60 ksi weld.

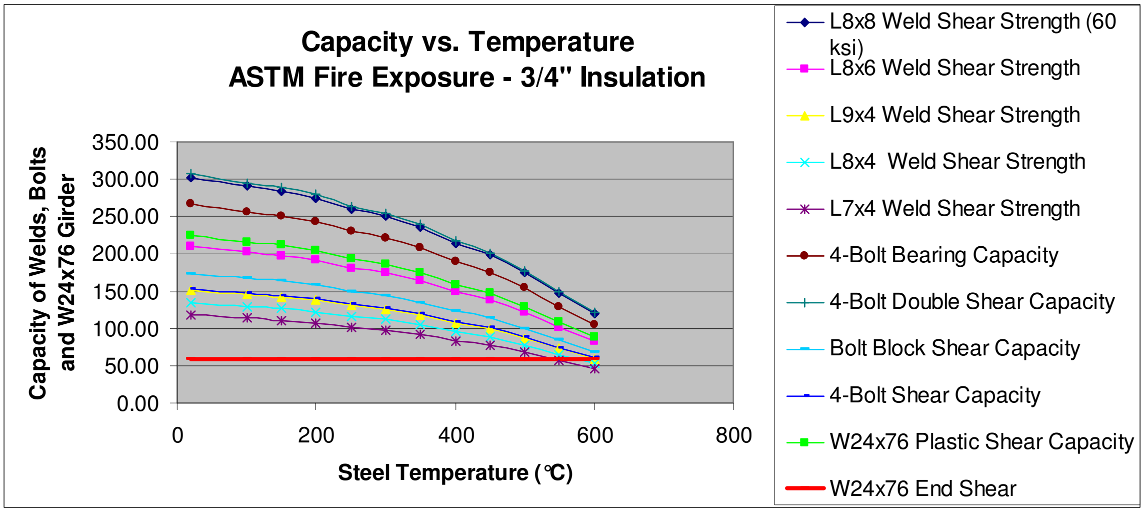

Figure 78 - Shear Capacities - L8x8x1/2" Angle & 1/4" fillet - 60 ksi weld metal

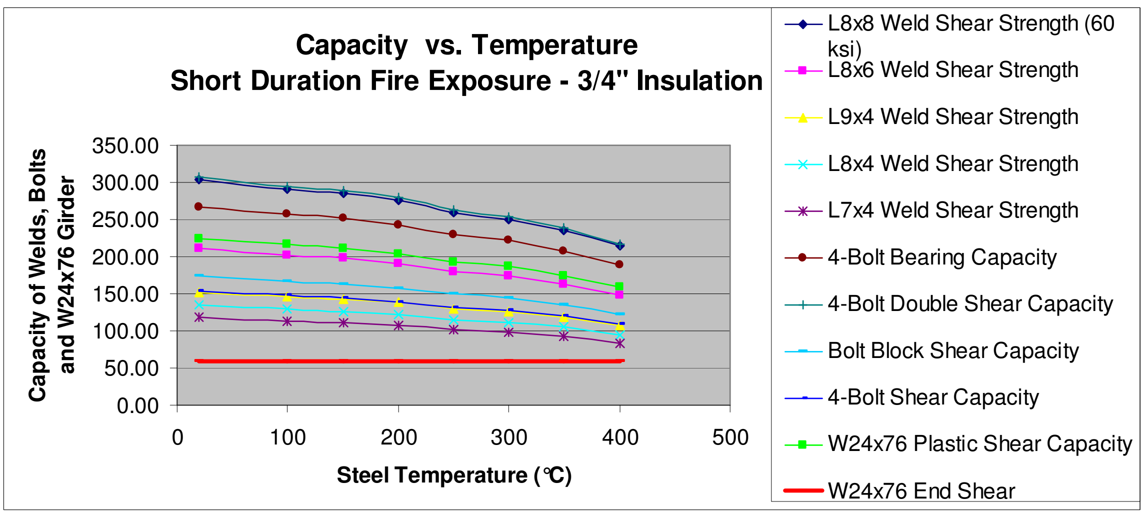

Figure 79 - Shear Capacities - L8x8x1/2" Angle & 1/4"fillet - 60 ksi weld metal

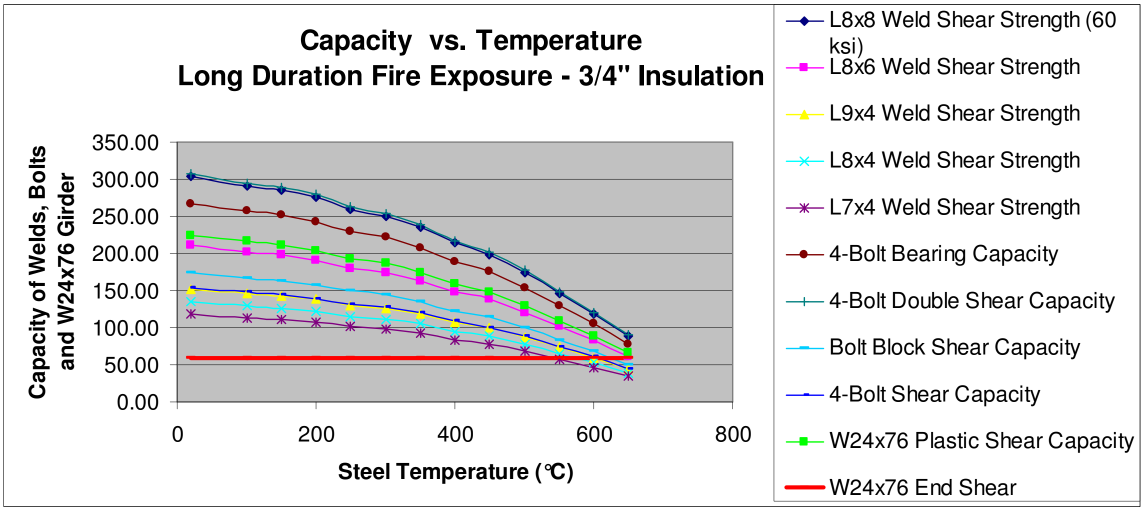

Figure 80 - Shear Capacities - L8x8x1/2" Angle & 1/4" fillet - 60 ksi weld metal

The three graphs below are the shear capacities of the structural elements for ¼" angles with a 3/16"- 60 ksi weld.

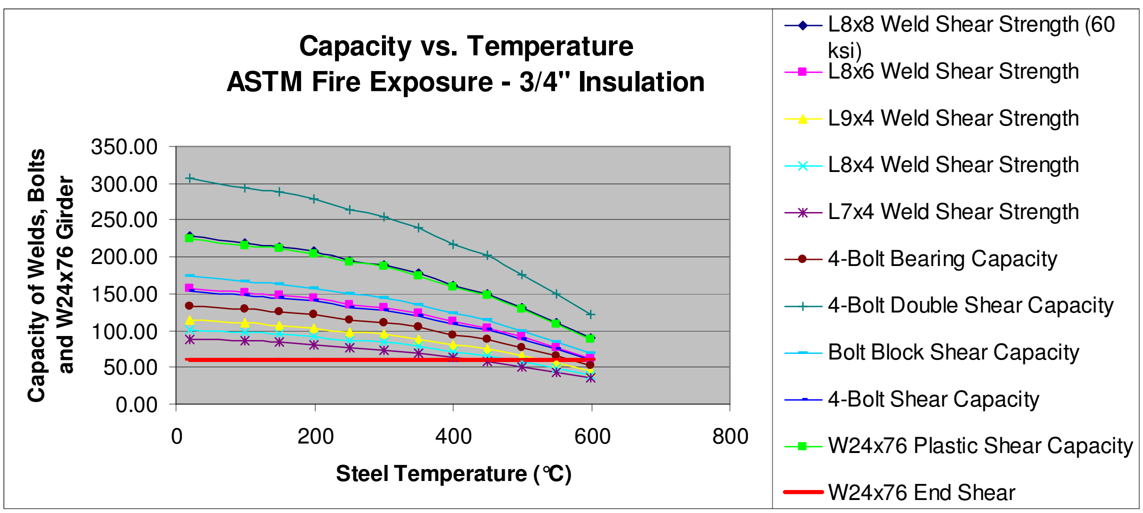

Figure 81 - Shear Capacities - L8x8x1/4" Angle & 3/16" fillet - 60 ksi weld metal

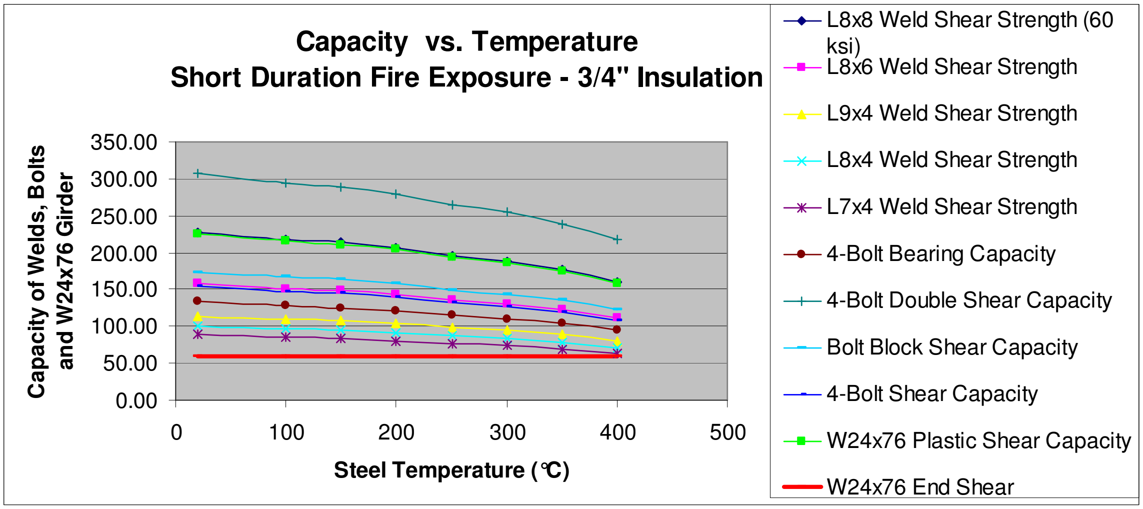

Figure 82 - Shear Capacities - L8x8x1/4" Angle & 3/16" fillet - 60 ksi weld metal

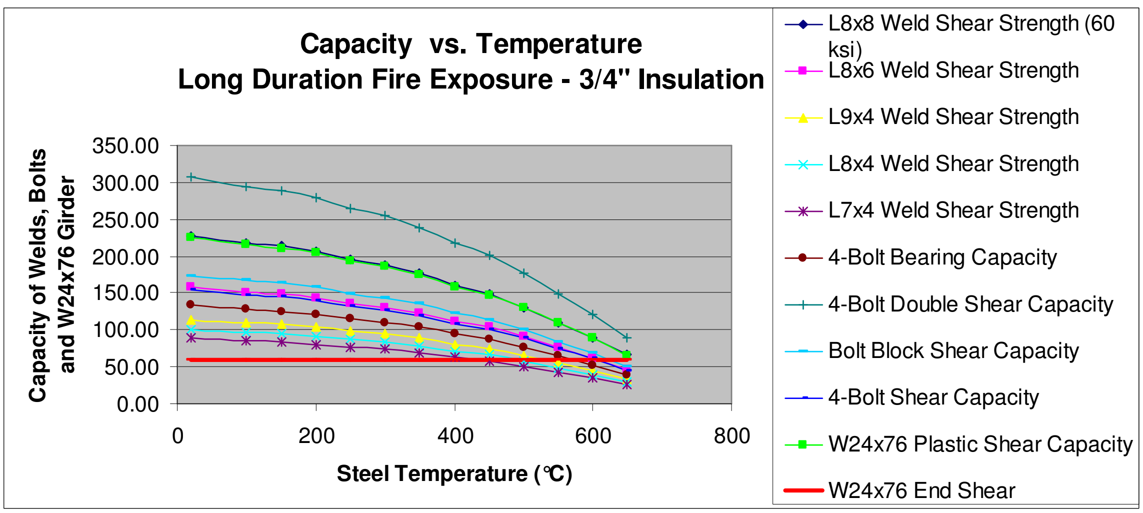

Figure 83 - Shear Capacities - L8x8x1/4" Angle & 3/16" fillet - 60 ksi weld metal

The final three graphs shown are the shear capacities of the structural elements for a ¼" angle and a 1/8"-60 ksi weld.

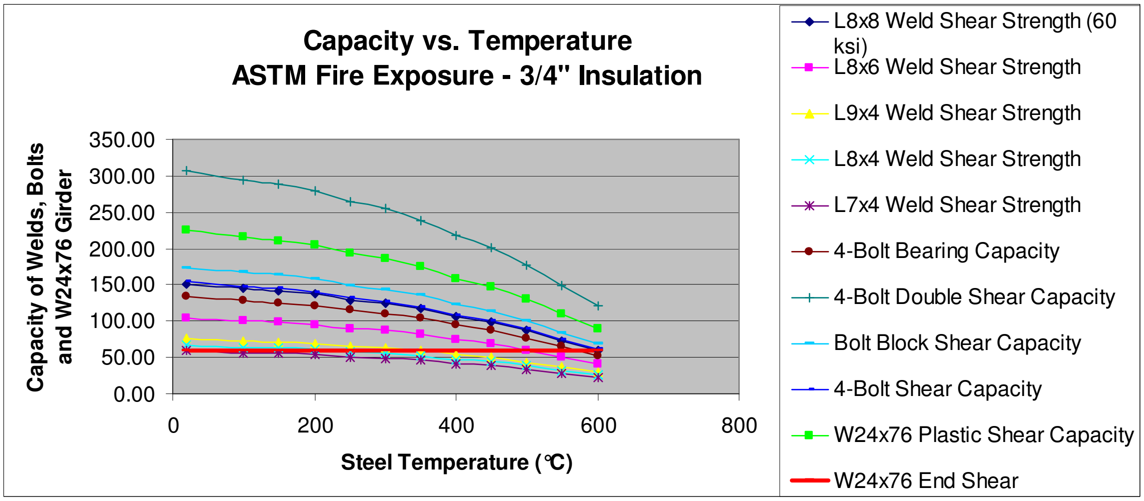

Figure 84 - Shear Capacities - L8x8x1/4" Angle & 1/8"fillet - 60 ksi weld metal

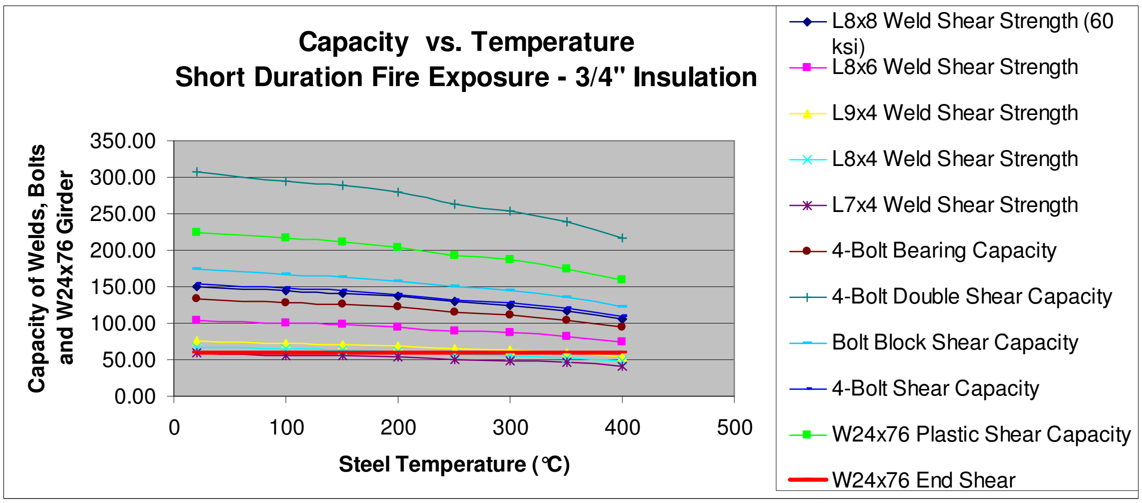

Figure 85- Shear Capacities - L8x8x1/4" Angle & 1/8" fillet - 60 ksi weld metal

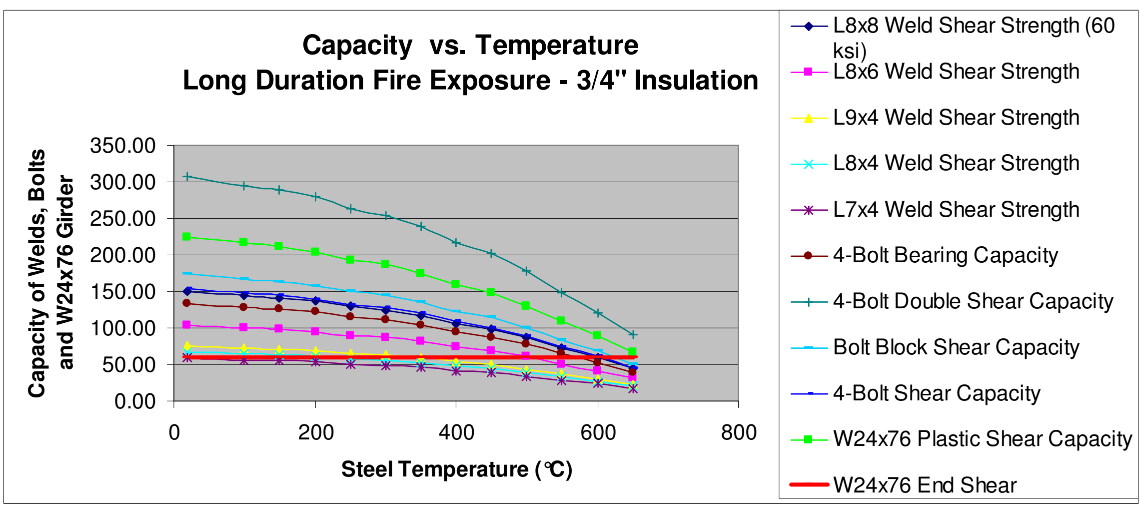

Figure 86 - Shear Capacities - L8x8x1/4" Angle & 1/8"fillet - 60 ksi weld metal

By looking at the twelve graphs above, the difference in strengths of variably sized ½" angles with 7/16" welds to variably sized ¼" angles with 1/8" welds is obvious. Below are tables summarizing the modes of failure by temperature for each of the cases illustrated.

Case # 1:

½" Angle – 7/16" Weld	600°C	625°C	650°C
ASTM	Bolt Shear		
Short			
Long	Bolt Shear	Block Shear	L7x4x1/2"

Case # 2:

½" Angle – ¼" Weld	575°C	585°C	600°C	610°C	650°C
ASTM	L7x4x1/2"	L8x4x1/2"	L8x8x1/2" L9x4x1/2"		
Short					
Long	L7x4x1/2"	L8x4x1/2"	L8x8x1/2" L9x4x1/2"	Block Shear	L8x6x1/2"

Case # 3:

¼" Angle 3/16 " Weld	425°C	533°C	500°C	575°C	600°C	625°C
ASTM	L7x4x1/4"	L9x4x1/4"	L8x4x1/4"	Bolt Bearing	L8x6x1/4" Bolt Shear	
Short						
Long	L7x4x1/4"	L9x4x1/4"	L8x4x1/4"	Bolt Bearing	L8x6x1/4" Bolt Shear	Block Shear

Case # 4:

¼ " Angle 1/8" Weld	50°C	200°C	320°C	500°C	585°C	600°C	625°C
ASTM	L7x4x1/4"	L8x4x1/4"	L9x4x1/4"	L8x6x1/4"	Bolt Bearing	L8x8x1/4" Bolt Shear	
Short	L7x4x1/4"	L8x4x1/4"	L9x4x1/4"				
Long	L7x4x1/4"	L8x4x1/4"	L9x4x1/4"	L8x6x1/4"	Bolt Bearing	L8x8x1/4" Bolt Shear	Block Shear

Table 11 - Failure Modes for Various Angles and Welds

Under normal conditions, all of the angles and welds investigated were adequate but as temperatures increased due to the three simulated fires, it is apparent how long each weld would effectively last. It is an assumption that connections degrade at the same rate as the members they connect; however, as can be seen above, this is not the case since the plastic shear capacity of the beam never reached the yielding end-shear strength of 59.18 kips.

The investigative report for this failure stated the:

"Girders and beams supporting the collapsed section of the floor slab were virtually straight and did not have the typical distortion such as bending, elongation, or twisting associated with steel frame members that have been exposed to excessive heat and high temperatures. In locations within the building, the welds that secured the clip angles to the columns broke, resulting in the failure of girder-to-column connections…" (Isner, 1986).

The reason for the welds breaking was not investigated through that report, but suggestions for the failure were offered, which included, effects of fire on welds, quality and adequacy of the welds, and the "possible effects of secondary and thermal stresses," (Isner, 1986).

Through the four cases illustrated above, the effect of fire on welds is clearly visible. Since the failure was attributed to breaking of the welds, cases two through four provide scenarios governed by failure of the end welds. It was stated that the floor system was not exposed to excessive temperature; case #4 demonstrates that weld failure may occur at temperatures as low as 50°C.

3.3.3 Discussion

From the analysis it was determined that the insulation thickness did not alter the results significantly. Changes in the angle size, weld thickness and strength however altered the outcomes greatly. As can be seen in Table 6 above, the ½" angle, with 7/16" weld, strength 60 ksi, would have failed by bolt shear at 600°C before any of the weld thicknesses tested failed. As the angle sizes decreased and the weld sizes decreased, failure of the welds became the governing mode of failure. As can be seen from Table 6, certain configurations of angles and welds could have contributed to failures at temperatures between 50°C and 320°C which are far less than the critical temperature of 600°C for steel beams.

The output from the analysis done for this case illustrates the potential for weld failure at relatively low temperatures. The prevailing notion is that beam failure would occur before connection failure due to the location and size (in terms of material) of the connections. However, the results presented above disprove that notion. To get a more realistic picture of what shear forces and moments were sustained by the welds, one could proceed with the RISA analysis and alter the strength of the columns with time as a function of temperature. One could also perform a finite element analysis for just the angles to determine the internal stresses in the member.

3.4 One New York Plaza

One New York Plaza was a 50-story office building, "the first 20 stories are approximately 222 feet by 286 feet and the next 30 stories (the tower section) are approximately 143 feet by 286 feet," (Powers, 1970). The building was constructed of a reinforced concrete core with an outer structural steel frame and a composite floor system. The beams, girders, and columns were all protected with a sprayed-on asbestos fiber, which was later found to have not adhered properly to the steel due to rust.

On August 5, 1970 a fire broke out on the 33rd floor, which was believed to have started in a telephone equipment room. The fire caused shear connections to fail and beams to drop onto girder flanges, resulting in a partial collapse of the 34th floor.

One New York Plaza was a case chosen to illustrate the "lessons learned" that were generated from this fire. While member collapses occurred due to the failing of end connections and lack of fireproofing, these were very localized and did not give way to a progressive collapse. The importance of this case is the recommendations that came from the report, some of which changed local law and practices, and some of which went unheeded and the consequences of such actions would be seen again.

There were fourteen recommendations that came from the investigative report of this fire. The suggestions made were for all buildings similar to One New York Plaza and not specific to just that building. The recommendations are listed below (Powers, 1970):

15. "The use of highly flammable foamed cushioning should be prohibited.
16. The total fire load of "fire resistive" buildings must be reduced or automatic sprinklers installed.
17. Wire (power and communication) in any part of an air conditioning system should be encased in metal conduit or ducts.
18. The protection of steel members in a really fire resistive building must be accomplished by materials that cannot be readily removed or damaged. It is apparent that sprayed fiber may not be universally applied to the proper thickness, that proper adhesion to steel may not take place and that the protection may be removed in many locations, such as at partitions, where ducts or wiring is run, and where clamps and brackets are attached.
19. Vertical flues in exterior walls between the skin and inner walls of partition should be cut off at each floor by a horizontal fire barrier with fire resistance equal to the floor.
20. Where openings through floors for air conditioning ducts are permitted, the duct should go directly to a non-combustible material in the duct passage.
21. Wiring connections through floors should be provided with thermal insulation to prevent transmission of heat thereby negating the fire resistance of the floor.
22. Air conditioning system should preferably be restricted to serving only one floor.

23. Automatic smoke detectors should be provided at each opening in the return air shaft unless the building is sprinklered.
24. Air supply for computer rooms should be from a location remote from the building air-conditioning intake and discharge, so that the computer rooms will not be subject to smoke from fires in other sections of the building.
25. Means should be provided for venting the building during a fire.
26. Elevator call buttons should not be of a type that will call and elevator to a floor because of heat, smoke or flames.
27. Prefire plans should be drawn up for all buildings. This should include procedures for notifying occupants, calling the fire department, routes of exiting form the various floors and protection of valuable equipment.
28. Special equipment for fire department use for operating windows, shutters, fans and elevators should be provided and a planned procedure for emergency operation of the air conditioning system should be formulated" (Powers, 1970).

In some respects, building design and construction has come a long way since this fire. Advancements in fire protection and compartmentalization have aided in the containment of fire. Buildings of a certain size and classification must now be equipped with automatic sprinklers by law. Also required by law are pre-fire plan evacuation routes, lighted exit signs, and other requirements that aid in the preservation of lives.

The three major recommendations that failed to initiate any change are numbers one, four and eight. While fire retardant material is more commonly used today, plush furniture and excess material continue to severely increase the fuel loads in buildings.

Due to cost, the means of insulating steel and air-conditioning a building have not changed. The issue of insulating steel in a manner that does not sacrifice the integrity of the steel if removed has been seen time and time again in the aftermath of One New York Plaza. Most recently are the cases of World Trade Center collapses. In the case of One New York Plaza, the steel beams were insulated with a spray-applied asbestos fiber, however due to rust on the original steel; the fiber did not adhere properly. Also, in places that the protection did adhere, it was removed for the installation of ducts, wiring, or other work done in the building. The fire began and quickly spread through openings in the ceiling where the tiles had been removed for wiring. The combination of the missing ceiling tiles and the lack of insulation allowed excessive heat to penetrate the steel. The temperatures reached resulted in bending and twisting of the steel beams and shear failure of the end connections. The integrity of spray-applied insulation and quality was again questioned, in almost the same words, by the 2002 FEMA World Trade Center Building Performance Study: Data Collection, Preliminary Observations, and Recommendations.

In the 1970's there was a large demand for sprinkler installation throughout high-rise buildings. Since then, it has been the trend to decrease the thickness of insulation if automatic sprinklers have been installed. However, the effectiveness of automatic sprinklers is not certain, as seen in WTC 5 when the sprinklers did not operate to control the fire. In many cases automatic sprinklers become inoperable, for example, when the metal heads fuse due to the heat generated by the fire or are designed incorrectly.

While it is more economical to reduce the insulation if adding automatic sprinklers, it may not be the best option from a structural standpoint. If the sprinklers are ineffective, then the structure is left exposed to the fire. Recommendations such as the ones presented above, as well as those from other failures due to fire should be kept in mind during design in order to design safer structures and avoid failure.

4 Discussion & Conclusions

The cases investigated above and their modes of failure were done so to provide structural engineers with information regarding failures caused by fire. Through the series of case investigations, the reader can see the original and alternative modes of failure. This manual was designed to be a teaching aid and to promote interest and discussion in this field of study.

There are several assumptions that must be noted when studying this manual. The first, and perhaps most important was the assumed mode of failure for each case. These were determined from the investigative reports of the failures studied.

Other assumptions made involved the heat transfer analyses. The analyses performed in this project to determine the steel temperatures and material properties (yield strength and modulus of elasticity) are based on the lumped mass heat transfer method. In order to use this approach, certain assumptions were made. The first assumption made was the temperature of the steel throughout the entire cross section was at the same temperature. This was done to simplify the analysis. This is based on the fact that steel is a good conductor of heat; however a constant temperature throughout a member is not generally the case under real fire conditions. Other assumptions made in these analyses were the values for the material properties of the steel and insulation, which can be seen in Tables 3 and 4.

Secondary analyses of the connections in the World Trade Center 5 and Alexis Nihon Plaza cases involved the use of RISA-2D structural analysis software. This software was used to determine the forces in the members of the framing systems for each case. For input into this program, some information was known, but not all, which led to assumptions of the boundary conditions, material properties, and loading of the members. One major assumption made was that as the steel girders in the composite systems degraded the columns maintained their original properties. While columns often have a thicker fire protection layer and lag the degradation of the other members, they do still degrade to some extent with respect to fire.

There are certain limitations that accompany the assumptions and analyses performed for the cases investigated. The first is the simplified equations used in the lumped mass heat transfer method. These equations do not consider the spread of heat through the steel. The values for the temperature change of steel according to time and gas temperature (calculated every 50°C) were then used to obtain the material properties of the steel. These equations, seen in Equations 6-9, are solely based on temperature, limited to 1000°C, and do not account for any losses that may occur in the steel. Since the critical temperature for most steel members is 593°C (Milke, 2002), determined from Figure 33, failure would likely occur before ever reaching 1000°C.

Another limitation encountered was the use of the RISA-2D software. The two versions of this software differ in the complexity of the input for each program. A more extensive knowledge of this program would have been useful in furthering analyses for the cases studied.

The last limitation encountered was the modes of failure determined from the investigative reports. Analyses were performed with the modes of failure in mind and geared towards those end results. If the failure modes had been undetermined, a much broader investigation would have had to be completed to determine various modes of failure.

In the case study manual developed, two major points have arisen. The first is the importance of insulation, which includes but is not limited to, the insulation material, its application, and the quality control issues that arise with insulation. As was discussed in the recommendations from the One New York Plaza investigative report, the application of spray-applied insulation material needs to be controlled or inspected in a manner as to ensure effectiveness. This point was again raised in the FEMA World Trade Center Building Performance Study: Data Collection, Preliminary Observations, and Recommendations 30 years later.

The second point was the need for an in-depth study of the behavior of connections.

> "The performance of connections seem to often determine whether collapse is localized or leads to a progressive collapse. In the standard fire tests of structural members, the member to be tested is wedged into a massibe restraining frame. No connections are involved. The issue of connection performance under fire exposure is critical to understanding building performance and should be a subject of further research," (Milke, 2002).

Three of the four cases examined suffered collapses due to connection failure. In the analysis of the World Trade Center 5, it was determined that the shear tab connections failed due to extreme tensile and shear (due to collapse loads) forces acting on the connections. The design of simple shear connections would not have included analysis for the secondary tensile forces developed during the fire. The second case examined, Alexis Nihon Plaza, investigated the capacities of weld and bolts when exposed to fire. It was found from the technical report of this failure that the floor system was not exposed to excessive heat from the fire. Analysis of the weld and bolts for various plate thicknesses revealed that it is possible for a sound design of welds could produce failure at relatively low temperatures. If structural engineers are to begin designing for fire conditions, guidelines must be established for determining acceptable temperatures and standards for calculating thermal forces.

Through the analyses presented, it has been demonstrated that it is possible to determine modes of failure for different framing systems using simplified equations and models. The results of these analyses allow for a better understanding of how different structural elements behave under various fire conditions. The analyses performed for this project are related to specific cases but can be applied generally to alternative cases, allowing for structural engineers to design for fire conditions using simplified models.

The 2005 AISC *Specification for Structural Steel Buildings, Appendix 4: Structural Design for Fire Condition* defines several different fires and methods by which analysis can be accomplished. Since this is the first time this section has appeared in the Specifications, there is much room for expansion. Alternatively, by this section appearing in the specification, many questions are raised with its implications. Currently, the NFPA is responsible for the fired protection systems that appear in their published codes, so the issue of responsibility is an issue. Also, if structural engineers are to assume this responsibility, their fees must increase, and peer reviewers must also learn the new design analysis techniques. Another issue that arises with designing for fire is the amount of material, which will increase the overall project cost for structures. Over the years buildings have become lighter and lighter, and by introducing the idea of designing for fire goes against the standards of design and construction as presently practiced. In order to move performance based design forward, the questions raised above must be answered.

If these questions are answered and designing proceeds in this area, there is much room for the development of useful material. By developing the available tools further and expanding upon them, a monograph, similar to the CRSI: Reinforced Concrete Fire Resistance book could be developed. A book of this sort could contain equations and methods for analysis, state of the art designs and sample designs, and analysis and discussion of failure cases.

Another direction for future work would be the development of furnace tests that simulate natural fires to get a more realistic idea of the behavior of systems under these conditions. Connections should also be tested under both the ASTM E-119 and natural fire exposures to determine their behavior based on tests rather than assumptions.

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Appendix B

Sample Calculations and Models

Sample Lumped Mass Heat Capacity Analysis Calculations

t (min)	Δt (sec)	Tg (°C)	Avg. Tg (°C)	Coefficient	ΔTs (°C)	Steel T (°C)
0	60	20	45	5.31845E-05	0.1	20
1	60	70	85.79	5.31845E-05	0.2	20.1
2	60	101.58	117.37	5.31845E-05	0.3	20.3
3	60	133.16	148.95	5.31845E-05	0.4	20.6
4	60	164.74	180.53	5.31845E-05	0.5	21.0
5	60	196.32	212.11	5.31845E-05	0.6	21.5
6	60	227.9	243.69	5.31845E-05	0.7	22.1
7	60	259.48	275.27	5.31845E-05	0.8	22.8
8	60	291.06	306.85	5.31845E-05	0.9	23.6
9	60	322.64	338.43	5.31845E-05	1.0	24.5
10	60	354.22	370.01	5.31845E-05	1.1	25.5
11	60	385.8	401.59	5.31845E-05	1.2	26.6
12	60	417.38	433.17	5.31845E-05	1.3	27.8
13	60	448.96	464.75	5.31845E-05	1.4	29.1
14	60	480.54	496.33	5.31845E-05	1.5	30.5
15	60	512.12	527.91	5.31845E-05	1.6	32.0
16	60	543.7	559.49	5.31845E-05	1.7	33.6
17	60	575.28	591.07	5.31845E-05	1.8	35.3
18	60	606.86	622.65	5.31845E-05	1.9	37.0
19	60	638.44	654.23	5.31845E-05	2.0	38.9
20	60	670.02	685.81	5.31845E-05	2.1	40.9
21	60	701.6	717.39	5.31845E-05	2.2	42.9
22	60	733.18	748.97	5.31845E-05	2.2	45.1
23	60	764.76	780.55	5.31845E-05	2.3	47.3
24	60	796.34	812.13	5.31845E-05	2.4	49.7
25	60	827.92	843.71	5.31845E-05	2.5	52.1
26	60	859.5	875.29	5.31845E-05	2.6	54.6
27	60	891.08	906.87	5.31845E-05	2.7	57.3
28	60	922.66	938.45	5.31845E-05	2.8	60.0
29	60	954.24	970.12	5.31845E-05	2.9	62.8
30	60	986	976	5.31845E-05	2.9	65.7
31	60	966	956	5.31845E-05	2.8	68.6
32	60	946	936	5.31845E-05	2.8	71.4
33	60	926	916	5.31845E-05	2.7	74.2
34	60	906	896	5.31845E-05	2.6	76.8
35	60	886	876	5.31845E-05	2.5	79.5
36	60	866	856	5.31845E-05	2.5	82.0
37	60	846	836	5.31845E-05	2.4	84.5
38	60	826	816	5.31845E-05	2.3	86.9
39	60	806	796	5.31845E-05	2.3	89.2
40	60	786	776	5.31845E-05	2.2	91.4
41	60	766	756	5.31845E-05	2.1	93.6
42	60	746	736	5.31845E-05	2.0	95.7
43	60	726	716	5.31845E-05	2.0	97.8
44	60	706	696	5.31845E-05	1.9	99.8
45	60	686	676	5.31845E-05	1.8	101.7
46	60	666	656	5.31845E-05	1.8	103.5
47	60	646	636	5.31845E-05	1.7	105.3
48	60	626	616	5.31845E-05	1.6	107.0

ASTM E-119		Fyo =	36		Eto =	29000	
			ti= 0.0			ti= 0.5	
t	Tg	Steel T	Fyt	Et	Steel T	Fyt	Et
(min)	(°C)	(°C)	(ksi)	(ksi)	(°C)	(ksi)	(ksi)
0	20	20.0	36	29000	20.0	36	29000
1	180	58.3	35.31458	28712	20.8	35.81267	28924
2	360	159.6	33.33343	27801	23.2	35.78589	28913
3	540	298.6	29.24472	25679	27.2	35.73827	28893
4	571	421.6	24.15268	22626	32.3	35.67644	28867
5	626	506.3	19.67307	19540	37.7	35.60685	28838
6	704	582.2	14.83674	15729	43.7	35.526	28803
7	719	644.1	10.77925	12057	50.1	35.43583	28765
8	734	683.5	8.732661	10050	56.6	35.34037	28723
9	749	711.3	7.498258	8783	63.1	35.23965	28680
10	764	732.9	6.63168	7866	69.8	35.13366	28633
11	779	751.4	5.95037	7128	76.5	35.02237	28584
12	794	768.2	5.371626	6490	83.3	34.90576	28532
13	803	782.7	4.900911	5963	90.1	34.78427	28478
14	812	794.6	4.534028	5547	97.0	34.65845	28421
15	821	805.1	4.222447	5190	103.9	34.5283	28362
16	828.4	814.5	3.953154	4879	110.8	34.39399	28300
17	835.8	822.9	3.718657	4606	117.7	34.25569	28236
18	843	830.8	3.5054	4356	124.6	34.11344	28170
19	849	838.1	3.31387	4130	131.5	33.96739	28102
20	855	844.7	3.142453	3926	138.4	33.8177	28032
21	862	851.3	2.976736	3729	145.3	33.66427	27959
22	867.2	857.7	2.82022	3541	152.2	33.50721	27884
23	872.8	863.6	2.6778	3370	159.1	33.34668	27807
24	878	869.2	2.543635	3208	165.9	33.18271	27728
25	882.6	874.5	2.420252	3058	172.8	33.01542	27647
26	888.7	879.9	2.298322	2909	179.6	32.84473	27563
27	892	884.9	2.185056	2771	186.4	32.67081	27478
28	896.4	889.3	2.08593	2649	193.2	32.49388	27390
29	900.8	893.8	1.988805	2530	199.9	32.31382	27300
30	905	898.1	1.894348	2413	206.7	32.13069	27208
31	908.8	902.3	1.804882	2302	213.4	31.94457	27114
32	912.6	906.3	1.720475	2198	220.0	31.75554	27017
33	916	910.1	1.641086	2099	226.7	31.56366	26919
34	919.8	913.9	1.564663	2003	233.3	31.36896	26819
35	923.6	917.6	1.488204	1908	239.9	31.17139	26716
36	927	921.3	1.414055	1815	246.4	30.97102	26611
37	930.4	924.8	1.343359	1726	253.0	30.76793	26504
38	933.8	928.3	1.274762	1640	259.5	30.56212	26395
39	937	931.7	1.208492	1556	266.0	30.35365	26284
40	940	935.0	1.145559	1477	272.4	30.14259	26170
41	943	938.1	1.085515	1400	278.8	29.92898	26055
42	946	941.2	1.027197	1326	285.2	29.71285	25937
43	948.8	944.1	0.970902	1255	291.5	29.49423	25817
44	951.6	947.0	0.916754	1186	297.8	29.27318	25695
45	954	949.8	0.865682	1121	304.1	29.04978	25571

ASTM E-119 Fire Exposure - Composite Floor			
1/2" Insulation	W18x50	W24x61	
Temperature	E	E	Deflection
20	28927.63	28927.63	0.324
100	28357.28	28360.04	0.331
150	27827.05	27919.52	0.337
200	27235.47	27255.91	0.345
250	26550.57	26537.44	0.354
300	25670.82	25660.87	0.366
350	24564.86	24556.97	0.382
400	23255.96	23270.24	0.404
450	21701.84	21697.84	0.432
500	19798.76	19801.07	0.474
550	17485.99	17483.07	0.537
600	14652.15	14646.32	0.641
650	11757.14		0.784
Short Duration Fire Exposure - Composite Floor			
1/2" Insulation	W18x50	W24x61	
Temperature	E	E	Deflection
20	28927.63	28927.63	0.324
100	28412.9	28363.08	0.33
150	27879.29	27871.69	0.337
200	27256.1	27250.84	0.344
250	26585.96	26614.61	0.353
300	25671.23	25755.96	0.366
350	24548.06	24615.23	0.382
400	23301.63	23242.71	0.403
450	21742.12		0.429
Long Duration Fire Exposure - Composite Floor			
1/2" Insulation	W18x50	W24x61	
Temperature	E	E	Deflection
20	28927.63	28927.63	0.324
100	28412.9	28363.08	0.33
150	27879.29	27919.52	0.337
200	27256.1	27250.84	0.344
250	26585.96	26514.06	0.353
300	25671.23	25755.96	0.366
350	24564.86	24615.23	0.382
400	23301.63	23242.71	0.403
450	21742.12	21573.34	0.432
500	19913.23	19751.21	0.472
550	17579.31	17509.75	0.534
600	14648.89	14638.68	0.641
650	10990.66		0.834

Sample Deflection Calculations

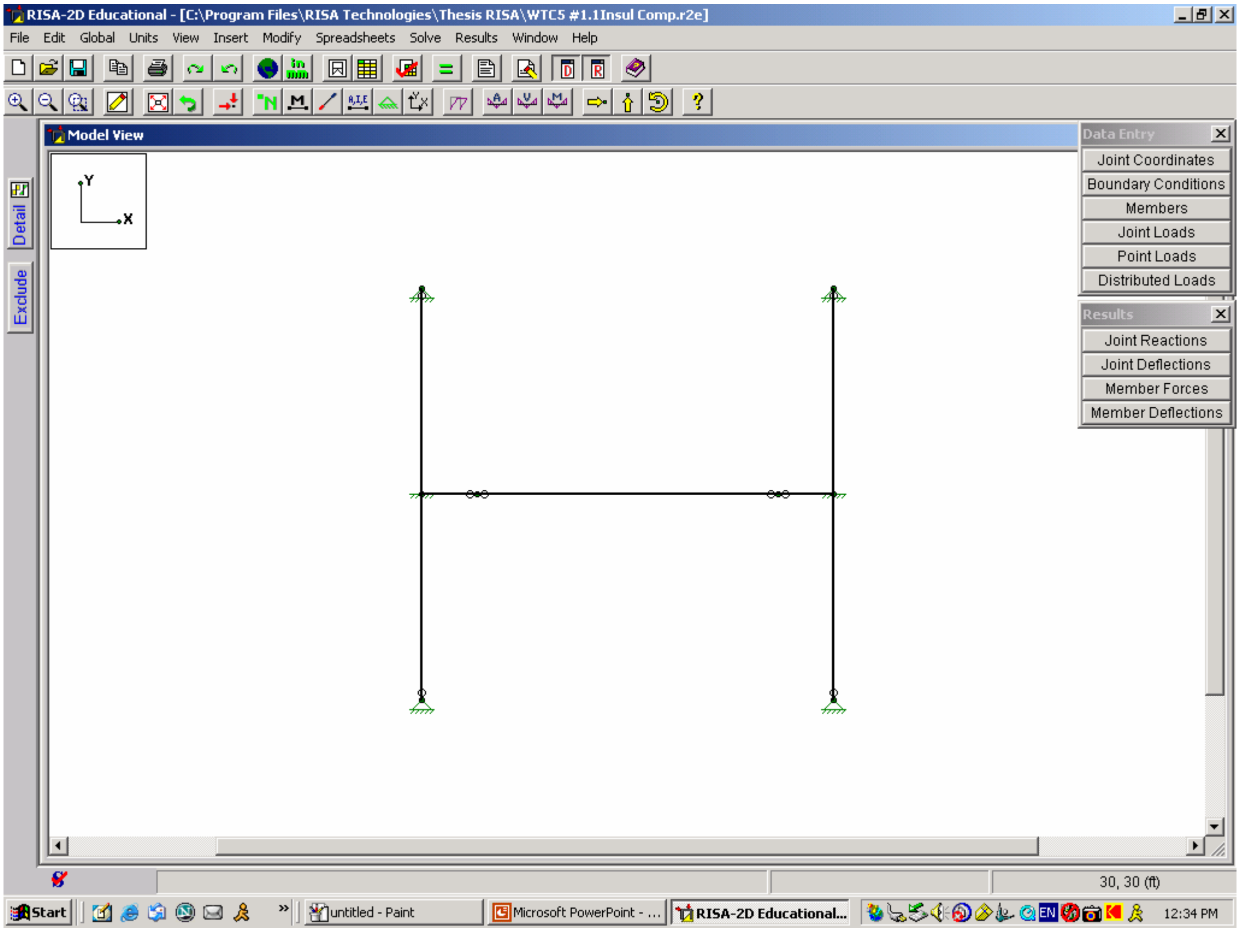
Steel Connections											
Fy	40	**W18x50**		**W24x61**		**A325 - 3/4" f Bolts**					
		Z_x	101	Z_x	152	db	0.75	Fub	120	Anv	2.63
		d	18	d	23	Fu	60	Ab	0.4418	Ant	1.125
		t_w	0.355	t_w	0.419	tw	0.375	Le	1.344	f	0.75
		w_u	4.04	w_u	4.04						
		L	22	L	4	**Shear**	**Shear**	**Tensile**	**Tensile**	**Block Shear**	
Temp.	**Fyt/Fy**	**Mp**	**Vp**	**Mp**	**Vp**	**Vu**	**Ru (D)**	**Ru**	**Ru**	**fRn**	
0	1	336.67	153.36	506.67	231.29	151.88	232.72	30.24	90.72	121.64	
20	0.99	333.30	151.83	501.60	228.98	150.36	230.39	29.94	89.81	120.42	
100	0.95	319.83	145.69	481.33	219.72	144.28	221.08	28.73	86.18	115.55	
150	0.93	313.10	142.62	471.20	215.10	141.24	216.43	28.12	84.37	113.12	
200	0.9	303.00	138.02	456.00	208.16	136.69	209.45	27.22	81.65	109.47	
250	0.85	286.17	130.36	430.67	196.59	129.09	197.81	25.70	77.11	103.39	
300	0.82	276.07	125.76	415.47	189.66	124.54	190.83	24.80	74.39	99.74	
350	0.77	259.23	118.09	390.13	178.09	116.94	179.19	23.28	69.85	93.66	
400	0.7	235.67	107.35	354.67	161.90	106.31	162.90	21.17	63.50	85.14	
450	0.65	218.83	99.68	329.33	150.34	98.72	151.27	19.66	58.97	79.06	
500	0.57	191.90	87.42	288.80	131.83	86.57	132.65	17.24	51.71	69.33	
550	0.48	161.60	73.61	243.20	111.02	72.90	111.71	14.52	43.55	58.38	
600	0.39	131.30	59.81	197.60	90.20	59.23	90.76	11.79	35.38	47.44	
650	0.29	97.63	44.47	146.93	67.07	44.04	67.49	8.77	26.31	35.27	
700	0.21	70.70	32.21	106.40	48.57	31.89	48.87	6.35	19.05	25.54	
750	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
800	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	

Sample Steel Connection Calculations

ASTM															
3/4" Insulation					24		20		17		16		15		
Steel Temp (°C)	**Fy**	**E**	**Vend**	**Mend**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**
20	35.82	28927.63	59.18	126.32	302.80	353.26	210.28	245.32	151.92	177.25	134.58	157.01	118.28	137.99
100	34.56	28520.43	59.18	125.30	290.56	338.99	201.78	235.41	145.79	170.08	129.14	150.66	113.50	132.42
150	33.46	28153.27	59.18	124.37	284.45	331.85	197.53	230.45	142.72	166.50	126.42	147.49	111.11	129.63
200	32.27	27277.95	59.18	122.10	275.27	321.15	191.16	223.02	138.11	161.13	122.34	142.73	107.53	125.45
250	30.98	26509.23	59.18	120.06	259.98	303.31	180.54	210.63	130.44	152.18	115.55	134.80	101.55	118.48
300	29.19	25648.64	59.17	117.71	250.80	292.60	174.17	203.20	125.84	146.81	111.47	130.05	97.97	114.30
350	27.30	24564.32	59.17	114.65	235.51	274.76	163.55	190.81	118.16	137.86	104.67	122.12	92.00	107.33
400	25.17	23273.27	59.17	110.85	214.10	249.78	148.68	173.46	107.42	125.32	95.16	111.01	83.63	97.57
450	22.76	21708.33	59.17	106.01	198.81	231.94	138.06	161.07	99.75	116.37	88.36	103.08	77.66	90.60
500	20.06	19821.37	59.17	99.80	174.34	203.39	121.07	141.25	87.47	102.05	77.48	90.40	68.10	79.45
550	16.99	17493.50	59.17	91.51	146.81	171.28	101.95	118.94	73.66	85.94	65.25	76.12	57.35	66.91
600	13.54	14616.52	59.16	80.17	119.28	139.16	82.84	96.64	59.85	69.82	53.02	61.85	46.60	54.36
Short Duration														
3/4" Insulation					24		20		17		16		15	
Steel Temp (°C)	**Fy**	**E**	**Vend**	**Mend**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**
20	35.82	28927.63	59.18	126.32	302.80	353.26	210.28	245.32	151.92	177.25	134.58	157.01	118.28	137.99
100	34.67	28424.79	59.18	125.06	290.56	338.99	201.78	235.41	145.79	170.08	129.14	150.66	113.50	132.42
150	33.56	27904.32	59.18	123.73	284.45	331.85	197.53	230.45	142.72	166.50	126.42	147.49	111.11	129.63
200	32.31	27297.19	59.18	122.15	275.27	321.15	191.16	223.02	138.11	161.13	122.34	142.73	107.53	125.45
250	30.93	26587.53	59.18	120.27	259.98	303.31	180.54	210.63	130.44	152.18	115.55	134.80	101.55	118.48
300	30.64	26436.03	59.18	119.86	250.80	292.60	174.17	203.20	125.84	146.81	111.47	130.05	97.97	114.30
350	27.46	24661.34	59.17	114.93	235.51	274.76	163.55	190.81	118.16	137.86	104.67	122.12	92.00	107.33
400	25.08	23213.93	59.17	110.67	214.10	249.78	148.68	173.46	107.42	125.32	95.16	111.01	83.63	97.57
Long Duration														
3/4" Insulation					24		20		17		16		15	
Steel Temp (°C)	**Fy**	**E**	**Vend**	**Mend**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**	**Vw_{60}**	**Vw_{70}**
20	35.82	28927.63	59.18	126.32	302.80	353.26	210.28	245.32	151.92	177.25	134.58	157.01	118.28	137.99
100	34.67	28424.79	59.18	125.06	290.56	338.99	201.78	235.41	145.79	170.08	129.14	150.66	113.50	132.42
150	33.56	27904.32	59.18	123.73	284.45	331.85	197.53	230.45	142.72	166.50	126.42	147.49	111.11	129.63
200	32.31	27297.19	59.18	122.15	275.27	321.15	191.16	223.02	138.11	161.13	122.34	142.73	107.53	125.45
250	30.93	26587.53	59.18	120.27	259.98	303.31	180.54	210.63	130.44	152.18	115.55	134.80	101.55	118.48
300	30.64	26436.03	59.18	119.86	250.80	292.60	174.17	203.20	125.84	146.81	111.47	130.05	97.97	114.30
350	27.46	24661.34	59.17	114.93	235.51	274.76	163.55	190.81	118.16	137.86	104.67	122.12	92.00	107.33
400	25.08	23213.93	59.17	110.67	214.10	249.78	148.68	173.46	107.42	125.32	95.16	111.01	83.63	97.57
450	22.83	21756.86	59.17	106.17	198.81	231.94	138.06	161.07	99.75	116.37	88.36	103.08	77.66	90.60
500	19.91	19715.49	59.17	99.44	174.34	203.39	121.07	141.25	87.47	102.05	77.48	90.40	68.10	79.45
550	17.00	17499.30	59.17	91.54	146.81	171.28	101.95	118.94	73.66	85.94	65.25	76.12	57.35	66.91
600	13.48	14562.69	59.16	79.94	119.28	139.16	82.84	96.64	59.85	69.82	53.02	61.85	46.60	54.36
650	9.76	11091.96	59.16	64.28	88.70	103.48	61.60	71.86	44.50	51.92	39.42	45.99	34.65	40.42

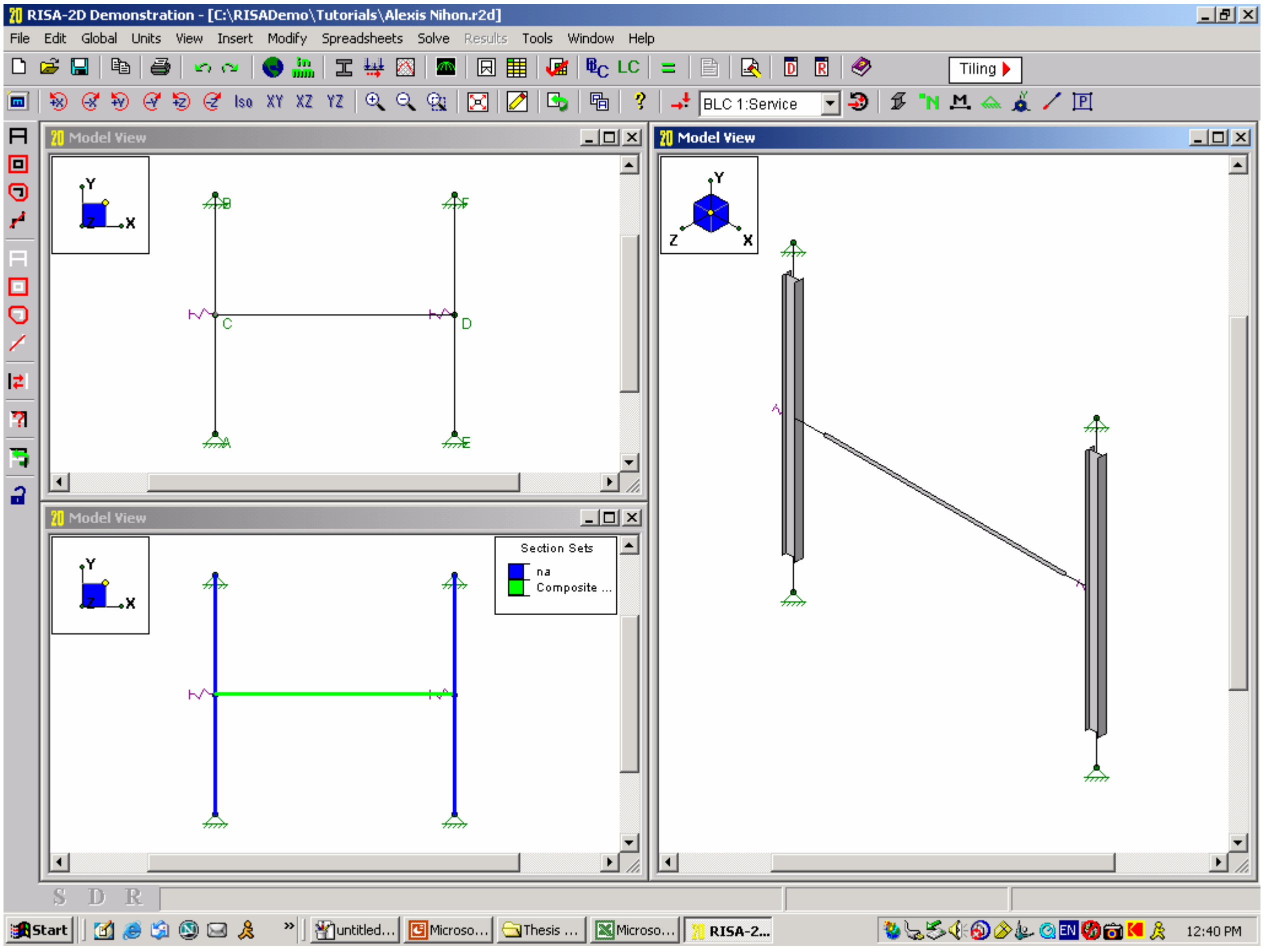
Sample Weld Calculations

RISA – 2D Sample Models
Educational & WebDemo



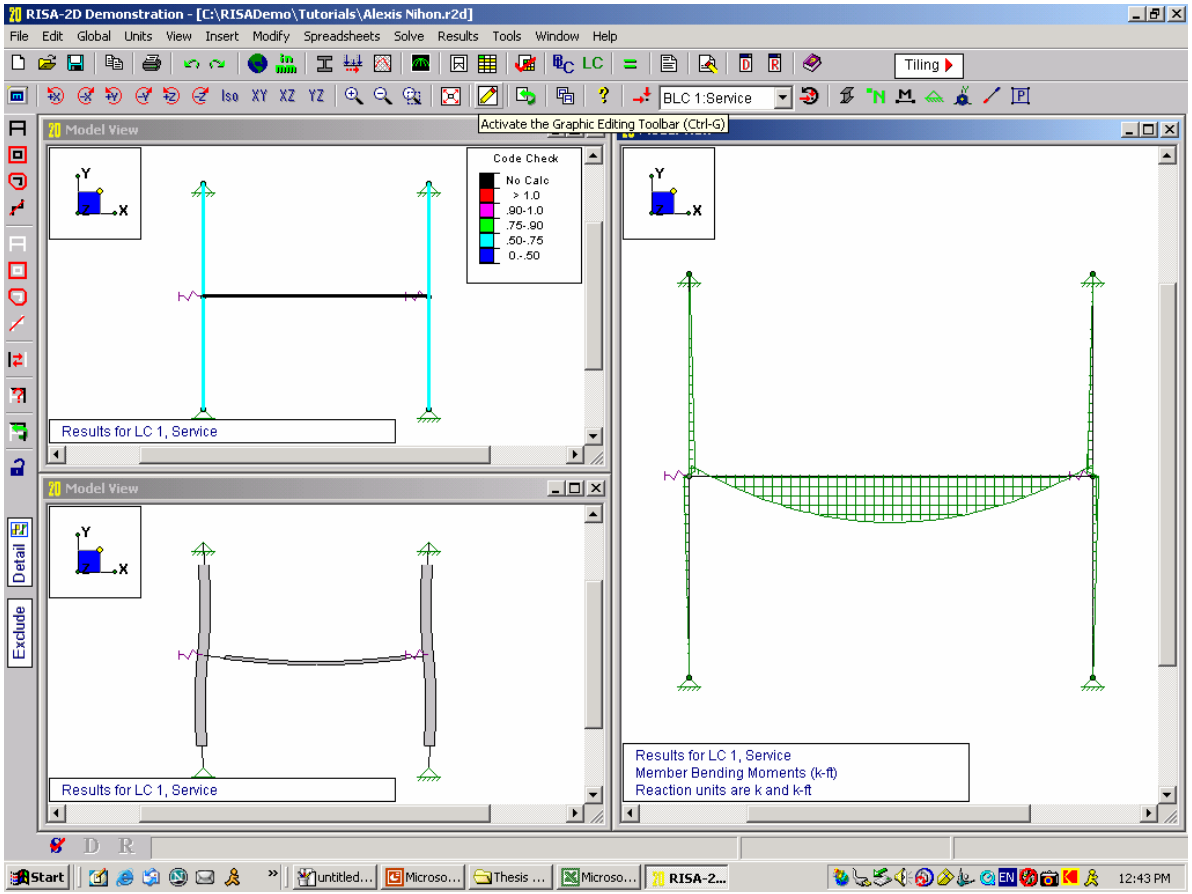
RISA-2D Educational

World Trade Center 5 – Original Framing System Model



RISA – 2D WebDemo

Alexis Nihon Plaza Framing Model



Alexis Nihon Plaza Sample Results