NATIONAL ENGINEERED LIGHTWEIGHT CONSTRUCTION FIRE RESEARCH PROJECT

TECHNICAL REPORT:

LITERATURE SEARCH
&
TECHNICAL ANALYSIS

by

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Foreword

This literature search and technical analysis was conducted for the Technical Advisory Committee to the National Engineered Lightweight Construction Fire Research Project to identify areas of concern, and to identify current gaps in documentation. As a basis for developing a fire test program it is, together with its extensive companion bibliography, a first step.

The National Engineered Lightweight Construction Fire Research Project was initiated in September, 1990 with the goal of documenting the fire performance of engineered lightweight construction and the performance of fire sprinklers in these assemblies. Phase I tasks include the identification of gaps in knowledge, and test planning.

For some years, there has been widespread concern among fire service, manufacturing, fire sprinkler and insurance communities regarding the fire performance of construction that relies more on strength of the engineering design than on mass. The concern is for misapplication, firefighter and occupant safety, roof or floor collapse, and fire suppression system adequacy. The concern is that there is inadequate documentation for many current practices, and misapplication of codes, resulting in inadequate safety factors,

The Research Foundation expresses gratitude to the author, Kirk Grundahl, P.E. The Foundation and the author thank the project's Technical Advisory Committee listed on the following page for their contributions in all respects: technical expertise, review, as well as the financial resources to conduct Phase I. Of course, the interpretation and opinions expressed are the author's and those of the authors of the literature cited, and project participation does not necessarily constitute a participant's endorsement of every statement in the report.

NATIONAL ENGINEERED LIGHTWEIGHT CONSTRUCTION PROJECT

Technical Advisory Committee

American Hotel & Motel Association
Boise Cascade Corp.
Building Officials and Code Administrators
Canadian Wood Council
Downey Fire Department
Factory Mutual Research Corp.
Fairfield Department of Public Safety
International Conference of Building Officials
Industrial Risk Insurers
L.A. County Fire Department
MacMillan Bloedel Ltd.
National Fire Sprinkler Association
National Forest Products Association
PFS Corporation
Qualtim Technologies
J. Gordon Routley, P.E.
Trus Joist MacMillan Co.
Underwriters Laboratories Inc.
Virginia Beach Fire Department
Willamette Industries
Wood Truss Council of America

Executive Summary

The overall objective of the National Fire Protection Research Foundation (NFPRF) National Engineered Lightweight Construction Fire Research Project is to define the actual fire performance characteristics of engineered components. The components examined in this study include: metal plate connected (MPC) wood trusses, MPC metal-web wood trusses, pin-end connected steel-web wood trusses, wooden I -joists, solid-sawn (e.g., 2 x 10) wood joists, composite wood joists, steel bar joists, and steel C joists.

Heavy timber trusses, bowstring timber trusses, heavy timber, and glue-laminated beams are discussed for additional information. They are not considered to be lightweight construction.

Phase I of this project involved performing a literature search to determine what relevant literature was available on this topic. A complete listing of the documents found in this search (approximately 2000 citations) may be obtained upon request from NFPRF.

During Phase II, literature pertinent to the topic was gathered and a technical analysis was prepared based on the readily available source documents. This analysis contains the following information:

- A discussion of the history of this topic, as well as the NFPRF project.
- A list of significant journal articles written about the fire performance of lightweight components and other related topics on fire performance. These articles are summarized and discussed by topic (e.g., trusses, steel, I -joists, glulam beams, timber trusses, heavy timber, connections, testing, fire ground tactics, etc.). The intent of this review is to accurately convey the concerns about fire performance, and to distill this information for further evaluation and discussion.
- A discussion of the statistics on losses due to fire. Covered are one- and two-family dwellings, apartments, non-residential fires, sprinkler performance, and civilian and firefighter injuries and fatalities. The intent of the statistics is to put the fire performance problem into perspective. The data can serve as a tool to help focus efforts that may be undertaken to enhance our ability to resolve performance concerns.
- A discussion of testing procedures and tests that have been used to assess the fire performance of the components under study. The testing is broken down as follows:

  - Unsheathed Assemblies.

  - Single Membrane Protected Assemblies.

  - Connections.

  - "Operation Breakthrough" Assemblies.

  - Coated Assemblies.

  In each of these areas, available tests are described, including title, author, sponsor, date, basic test description, test methods, data collected, and conclusions. Additional commentary is given after each test description to provide additional insight on the test method or other factors surrounding the test. A table summarizing all tests is included within each category listed above.

- A description of the sprinkler testing that has been performed primarily on engineered wood components. A detailed outline of the test specifics is provided, along with commentary on the tests performed.
- A discussion of requirements of the major U.S. model building codes: Uniform Building Code, National Building Code, and the Standard Building Code. The current local and model code development environment is also discussed.
- The information presented in each of the above topic areas is thoroughly discussed and evaluated. This includes a discussion of the literature that evaluates concepts presented in professional fire service journal articles, provides discussion of statistics, looks at failure modes of the various tested assemblies, and summarizes tests that have been performed. A series of test data summary tables identify where test data exists and where test data is not available.
- Conclusions based on the preceding information are developed. The major conclusions reached from this analysis are:
    - Lightweight building components are used extensively as structural members today, and the trend is for greater utilization in the future. Learning as much as possible about their structural and fire performance will only enhance firefighting safety .
    - Standardized test procedures and performance acceptance criteria must be developed, primarily to assist with determining modes of failure and warning signals prior to failure, and to support firefighting tactics, for the following areas:
        - Fire endurance performance of unsheathed lightweight building component assemblies.
        - Fire endurance performance of lightweight building component assemblies when a concealed space is created by their application.
        - Fire endurance performance of lightweight building component assemblies when sprinkler systems are incorporated.
    - There is a need for education and training in the following areas:
        - Engineering principles that apply to lightweight building components.
        - Explanation of the fire performance of lightweight building components.
        - Explanation of fire endurance testing procedures, and tables that help explain what the results from the testing performed mean.
        - Explanation of the use of mathematical fire endurance models as they are developed for lightweight building components.
        - The importance of code-conforming construction, and how violations of fire- and draftstopping influence the fire performance of lightweight building components.

- Strategy and tactics that are developed for fighting fires in buildings that employ lightweight building components. This includes developments based on current knowledge, as well as new knowledge gained through testing and experience.
- Develop the database technology that would aid pre-fire planning. This could then be expanded to provide detailed information on the performance of lightweight building components when fires have occurred in buildings that use them.

Appendix A gives a brief biographical sketch of the major authors cited in Chapter 2.

Appendix B is a glossary of related terms.

Appendix C reviews the comparative risk statistics.

Appendix D contains information pertaining to the bibliography of articles generated by the literature search.

Acknowledgments

This report is the culmination of discussion and guidance from the National Fire Protection Research Foundation Technical Advisory Committee. It represents the first time that the issues and test data surrounding the fire performance of lightweight components has been compiled into one document and evaluated. The purpose of this is to lay a factual foundation for this evaluation and provide the ability to set future direction on the issues surrounding lightweight component fire endurance performance. This project has been extremely difficult, due to the breadth of the issues surrounding this performance. The encouragement, feedback, guidance and commentary of the entire TAC is responsible for the finished product. In particular, we would like to thank those who took the time to make detailed comments on all the drafts prior to this completed document. They include:

Glenn Corbett	San Antonio Fire Department
Bob Glowinski	National Forest Products Association
Dennis Lockard	California Fire Chiefs
Alan Lambuth	Boise Cascade Corporation
Bob Berhinig	Underwriters Laboratories Inc.
Joe Piscione	Trus Joist MacMillan
Gordon Routley	Tridata
Tom Frost	Building Officials and Code Administrators International
Dick Davis	Factory Mutual Research Service Bureau
Dennis McCreary	International Conference of Building Officials

A special word of thanks must go to Glenn Corbett, Dennis Lockard, and Gordon Routley, for doing such a good job of providing their perspective and detailing the information that needed to be included in the document for relevance, accuracy, and clarity. We are particularly appreciative of Glenn and Dennis, who provided valuable insights, support, and encouragement.

The author also appreciates the inspiration and support provided by the Foundation and the administrators of this project, Rick Mulhaupt and Rich Bielen, through the travails that were encountered.

It is our fervent hope that this document will serve the general public's long-term interest in providing for the safe fire performance of lightweight component construction.

Kirk Grundahl
Jay T. Edgar

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4-4.4 Reference: "Report on a Fire Endurance Test of a Floor and Ceiling Construction, " UL File R6946-1, Underwriters' Laboratories, Incorporated, February, 1972 (Unpublished)..142

4-4.5 Reference: Son, B.C., "Fire Endurance Test of a Steel Sandwich Panel Floor Construction," NBSIR 73-164, National Bureau of Standards, April, 1973 143

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Chapter 1: Concern Over Fire Performance of Lightweight Building Component Construction

1.1 Introduction

Over the last several years, articles written in professional fire service journals have been warning of poor fire performance in buildings that are constructed with lightweight construction components. This, in turn, has lead to a very public debate on what the actual performance of these components is under fire exposure conditions. Given the disparate views on the fire performance of these structural elements, a reconciliation of the varying opinions on this issue is needed. Therefore, the National Fire Protection Research Foundation (NFPRF) undertook a project, called the **National Engineered Lightweight Construction Fire Research Project.** For this project, a Technical Advisory Committee (TAC) was created from organizations and individuals interested in the fire performance of lightweight building construction. This committee is chaired by J. Gordon Routley, P.E., a fire protection engineer and private fire service consultant. The organizations and individuals that make it up are listed below:

Participants

American Hotel & Motel Association
Boise Cascade
Building Officials and Code Administrators, International
California/Western Fire Chiefs Association
Canadian Wood Council
Downey, CA, Fire Department
Factory Mutual Research Corporation
Fairfield, CA, Department of Public Safety
Gordon Routley
Industrial Risk Insurers
International Conference of Building Officials
Los Angeles County Fire Department
National Fire Protection Association - Western Office
National Fire Sprinkler Association
National Forest Products Association
PFS Corporation
Qualtim Technologies International
Reedy Creek Improvement District
Schirmer Engineering Corporation
Trus Joist MacMillan
Underwriters Laboratories Inc.
Virginia Beach, VA Fire Department
Willamette Industries
Wood Truss Council of America

The first phase of the project gathered readily available, relevant literature on this topic to determine the documented understanding of the fire performance of these construction elements. A copy of this comprehensive search is available from the NFPRF. The second phase of the project is a review and analysis of the current understanding defined by this literature. This review and analysis forms the basis of this report.

1.2 History

A heightened professional concern over the performance of lightweight component construction was precipitated by the publication of Dr. Erwin L. Schaffer's article, "How Well Do Trusses Really Perform During a Fire?", in the March/April, 1988 edition of **Fire Journal** and a letter by Mr. Roger Montgomery, of Montgomery Builders Supply, Inc., to **Firehouse Magazine** in June, 1989. In summary, Montgomery expressed concern over the emotional nature of the fire service articles appearing in the press. He also noted that the Hackensack, New Jersey fire (where five firefighters lost their lives) did not involve metal plate connected (MPC) truss construction and should not be categorized by the fire service as performing in a fire the same as the heavy timber bowstring trusses in the automobile dealership. Schaffer's analysis claimed that comparative large-scale ASTM E119 fire testing and engineering analysis suggested that the fire performance of trusses *may* be equivalent to that of joist/rafter assemblies. He also stated that testing indicates truss assemblies give warning by deflecting substantially, and there is often flame-through of the sheathing near failure. These conditions should provide firefighters with sufficient warning of impending collapse.

This elicited a significant and varied response from the fire safety community, including the following published articles:

- "Are Wood Trusses Good for Your Health? The Safety Issue of Lightweight Wood Truss Floor Assemblies Provides Controversy," Francis L. Brannigan, **Fire Engineering**, June 1988.
- "Lightweight Wood Truss Floor Construction: A Fire Lesson," Glenn Corbett, **Fire Engineering**, July 1988.
- "How Wood Trusses Perform During a Real Fire," J. Gordon Routley, **Fire Journal**, January/February 1989.
- "A Primer on Truss Roofs. Why Truss Roofs are Hazardous for Firefighters," Francis L. Brannigan, **Firehouse**, March 1989.
- "Response to a Truss Manufacturer: The Dangers of Truss Construction Pointed Out, Again" (Due to Mr. Montgomery's letter), Francis L. Brannigan, **Firehouse**, June 1989.

While this debate provided an opportunity for public expression of the varying viewpoints, identification of specific concerns and possible solutions was not fully developed. As a result, in the spring of 1990, Qualtim Technologies International took the initiative to contact several prominent fire service members, engineered wood products manufacturers, and building regulators to determine their interest in developing an ad hoc group to discuss these issues. This led to the formation of an ad hoc committee, and a survey was sent out to gain a sense for current thought on this issue. Some pertinent comments from this survey on what was needed include:

Tom Brennan, **Fire Engineering Magazine**, answering, "What information do firefighters need from the engineered products group to better do their job?":

> *Just as you [the ad hoc committee] have started DIALOGUE AND CONCERN AND COOPERATION!!!*

Professor Bruce E. Cutter, University of Missouri, also Captain of Boone County Fire Protection District, answering, "What information do firefighters need from the engineered products group to better do their job?":

> *A pro-active approach to the problem is needed from both sides. We need to become advocates and supporters of early fire detection and suppression devices in both commercial and residential construction. We both need to participate in well-planned and documented studies that will examine some of the concerns the fire service has, and then make the results and recommendations known in both circles—fire service and truss manufacturers. Above all, we need to work together because neither the fire service nor the truss industry are going to go away.*

Robert Glowinski, National Forest Products Association, answering, "What are the short-term efforts that we need to undertake immediately to foster better fire safety?":

> *Establish ongoing dialogue between the fire service and engineered wood products industry.*

John Mittendorf, Los Angeles Fire Department, answering, "What are the short-term efforts that we need to undertake immediately to foster better fire safety?":

> *Continuing efforts of 'round table discussion.' Live meetings with an advance agenda.*

J. Gordon Routley, P.E., then of the Shreveport Fire Department, answering, "What should the long-term focus and goals for this particular group be?":

> *We need to develop educational material on all information for the fire service and designers.*

On June 27, 1990, at the Forest Products Research Society's 44th Annual Meeting, a special session was held on the Fire Performance of Light Frame Wood Structures. Speakers presented diverse papers on the following topics:

"Hazards of Fire Fighting in Light Frame Wood Structures," Francis L. Brannigan, Author/Lecturer, Port Republic, Maryland

"Truss Industry Response to Concerns of Building Industry Regarding Fire Performance of Light Frame Wood Construction," Kirk Grundahl, P.E., Founder, Qualtim Technologies International, Madison, Wisconsin, representing the Wood Truss Council of America.

"Tests to Improve Fire Safety of Structures Built Using Wood I-Beams," Joseph R. Piscione, P.E., Manager, Product Acceptance, Trus Joist Corporation, Boise, Idaho.

"Design and Use of Fire Sprinkler Systems to Suppress Flame Spread and Enhance Performance of Wood Frame Structures," Russell P. Fleming, P.E., Vice President, Engineering, National Fire Sprinkler Association, Patterson, New York.

"Model Building Codes and Fire Protection for Light Frame Wood Construction," J. Robert Nelson, P.E., Senior Vice President, PFS Corporation, Los Angeles, California.

Subsequent to this meeting, the NFPRF became interested in providing a forum for discussion of this issue. This led to a planning meeting on September 18, 1990, at the National Fire Protection Association's (NFiPA) Western Regional Office in Ontario, California. At this meeting it was concluded that a research project for lightweight component construction was needed. A project proposal was developed and sponsorship sought to undertake a literature review and technical analysis.

The rest of this report is a culmination of discussions and direction provided by the NFPRF TAC listed above. The document will cover fire service concerns found in the literature, statistics on fire performance, fire endurance testing performed on lightweight building components, a general overview of code requirements for these components, and will end with a discussion, conclusions, and recommendations section.

Chapter 2: Literature Review

2.1 Firefighting Articles

The literature search discussed in **Chapter 1** yielded a variety of articles related to lightweight component construction. The following articles were written by the fire safety community and are pertinent to the issue of fire performance of lightweight building construction. A chronological listing of these articles provides an indication of when the problem was first recognized, and when broad-based concern became apparent.

2.1.1 Before 1970

11/1/58, "Firemen Fear Floor Collapse," **Fire Engineering**.

2.1.2 1970 - 1979

7/1/70, "Building Weaknesses—Do You Know Them?", Brannigan, F.L., **Fire Command**.

4/1/71, "Collapse Danger of Roofs with Light Weight Wood Trusses," Brannigan, F.L., **Fire Engineering**.

11/1/71, "Three Firemen Hurt as Canopy Collapses," Varner, B., **Fire Engineering**.

3/1/73, "Built to Collapse," Brannigan, F.L., **Fire Chief**.

1/1/74, "A Field Study of Non Fire-Resistive Multiple Dwelling Fires," Brannigan, F.L., National Bureau of Standards.

2/1/75, "Recognizing the Probability of Building Collapse," Cruthers, F., **Fire Engineering**.

11/1/76, "Fire Feeds on Design Weakness," Nailen, R.L., **Fire Engineering**.

5/1/77, "Dangers of Steel Bar Joists and Noncombustible Buildings," Sylvia, D., **Fire Engineering**.

10/1/78, "Design for Disaster," Brannigan, F.L., **Fire Command**.

3/1/79, "Non-Combustible Buildings—Death Traps for Fire Fighters," Sylvia, D., **Fire Engineering**.

2.1.3 1980-1985 (6 years)

2/1/81, "Firefighter Dies in Fall Through Roof," Dektar, C., **Fire Engineering**.

3/1/81, "Educate Architects," Brannigan, F.L., **Fire Engineering**.

3/1/81, "Enforce Fire Stopping Rules," Brannigan, F.L., **Fire Engineering**.

9/1/81, "Predicting Building Collapse," Dunn, V., **Firehouse**.

1/1/82, "Appendix A: The Hotel Vendome Fire. In Building Construction for the Fire Service, 2nd Ed.," Brannigan, F.L., National Fire Protection Association.

1/1/82, "Lightweight Construction Tests Open Fire Service Eyes to Special Hazards," Mittendorf, J., **Western Fire Journal**.

9/1/82, "Two Firefighters Killed Trying to Ventilate Roof," Ludford, L., **Fire Engineering**.

12/1/83, "Collapse Dangers of Timber Truss Roofs," Dunn, V., **Firehouse**.

1/1/84, "Take the Surprise Out of Building Collapse," Brennan, T., **Fire Engineering**.

1/1/84, "We Have a Roof Cave In," Fekete, **Fire Command**.

1/1/84, "Lightweight Building Construction Helps Prevent a Major Disaster," Jones, J.L., **Fire Engineering**.

1/1/84, "Truss Fire and Collapse," **WNYF**.

2/1/84, "How Many Disasters Do We Need?", Brennan, T., **Fire Engineering**.

5/1/84, "Don't Hit the Steel—A Myth," Brannigan, F.L., **Fire Engineering**.

9/1/84, "Void Spaces (Training Notebook)," Brannigan, F.L., **Fire Engineering**.

10/1/84, "Floor Collapse in Residential Structures," Dunn, V., **Firehouse**.

11/1/84, "Chesterfield VA Church had Lightweight Roof Trusses," **Fire Command**.

2/1/85, "Operating on Steel Open-Web Bar-Joist Roofs," Dunn, V., **Firehouse**.

4/1/85, "Building Construction: Firefighting Problems and Structural Hazards," Dunn, V., **Firehouse**.

6/1/85, "Firefighting on Sloped Peaked Roofs," Dunn, V., **Firehouse**.

8/1/85, "Beware the Truss," Brannigan, F.L., **Fire Engineering**.

8/1/85, "Collapse Analysis and Safety Precautions," Dunn, V., **Firehouse**.

11/1/85, "Trusses I," Brannigan, F.L., **ISFSI Instructograms**.

12/1/85, "Trusses II," Brannigan, F.L., **ISFSI Instructograms**.

12/1/85, "Firefighting in Wood-Frame Buildings," Dunn, V., **Firehouse**.

2.1.4 1986 - 1989 (4 years)

2/1/86, "Built Like a Brick Outhouse—Or is it?", Brannigan, F.L., **Fire Engineering**.

6/1/86, "The Mything Link in Fire Protection," Brannigan, F.L., **Fire Engineering**.

9/1/86, "Know Your Roof," Brannigan, F.L., **Fire Engineering**.

9/1/86, "Truss Collapse: Final Report (Lessons learned from the fatal Waldbaum's fire)," Dunn, V., **Firehouse**.

12/1/86, "Hazards of Lightweight Wood Truss Construction," Dunn, V., **Firehouse**.

3/1/87, "Ceilings and Suspended Loads," Brannigan, F.L., **Fire Engineering**.

4/1/87, "Wooden Structures High in the Sky," Brannigan, F.L., **Fire Engineering**.

9/10/87, "Trusses Suspect in Collapse," **Engineering News Record**.

1/1/88, "The Quick Collapse of a 'Slow Burner'," Comer, W.J., **Fire Engineering**.

3/1/88, "The Metal Deck Roof Debate," Brannigan, F.L., Mittendorf, J., **Fire Engineering**.

6/1/88, "Are Wood Trusses Good for Your Health?", Brannigan, F.L., **Fire Engineering**.

7/1/88, "Lightweight Wood Truss Floor Construction: A Fire Lesson," Corbett, G.P., **Fire Engineering**.

7/1/88, "Joist-Rafter versus Lightweight Wood Truss," Mittendorf, J., **Fire Engineering**.

8/1/88, "Truss Construction Claims More Lives," Brennan, T., **Fire Engineering**.

9/1/88, Letter to the editor, Brannigan, F.L., **Fire Journal**.

9/1/88, "More Dangerous Myths," Brannigan, F.L., **Fire Engineering**.

9/1/88, "New Jersey's Darkest Hour," **Firehouse**.

9/1/88, "Trusses Can Kill," **Firehouse**.

10/1/88, "Five Fall in Hackensack," Corbett, G.P., **Fire Engineering**.

1/1/89, "How Wood Trusses Perform During a Real Fire," Routley, J.G., **Fire Journal**.

3/1/89, "Are Wood Trusses Good for Your Health?", Brannigan, F.L., **The Voice**.

3/1/89, "More Dangerous Myths," Brannigan, F.L., **Fire Engineering**.

3/1/89, "A Primer on Truss Roofs," Brannigan, F.L., **Firehouse**.

3/1/89, "Light Weight Truss Construction Gives Up More Lessons," Kurzeja, W., **Fire Engineering**.

4/1/89, "Dangers of Operating Above a Fire," Dunn, V., **Fire Engineering**.

5/1/89, "Fire Loss Management Series. Part 2: Why Can't We Convince Them?", Brannigan, F.L., **Fire Engineering**.

5/1/89, "Tragedy Knows No Boundary," **Firehouse**.

6/1/89, "Ties That Bind," Brannigan, F.L., **Fire Engineering**.

6/1/89, "Response to a Truss Manufacturer. The Dangers of Truss Construction Pointed out, Again," Brannigan, F.L., **Firehouse**.

7/1/89, "Orange County Fatal Fire: Investigation and Analysis," Orange County Fire and Rescue Division Investigation, **Fire Engineering**.

8/1/89, "Hazards of Truss Floors, part 1," Brannigan, F.L., **Firehouse**.

8/1/89, "Truss Roof Collapse" (video), Dunn, V.

2.1.5 1990 - Present

3/1/90, "The Peaked Roof," Dunn, V., **Fire Engineering**.

5/1/91, "Lightweight Wood Trusses: More to Consider," Manny, W.F., **Fire Engineering**.

5/1/91, "The Timber Truss: Two Points of View," Mittendorf, J.W., Brannigan, F.L., **Fire Engineering**.

12/15/91, "Structural Collapse: Pinpointing the Dangers," Dunn, V., **Firehouse**.

The foregoing list is evidence that the professional concern over this issue came to the fore in 1984, and has progressed from there. The movement appears to have been led by Mr. Francis L. Brannigan, a fire protection educator and author from Maryland, and Mr. Vincent Dunn, Deputy Chief of the New York City Fire Department. A brief biography of each of these, as well as several other authors, is found in **Appendix B**.

The literature search revealed that the concern with lightweight component construction fire performance is, for the most part, concentrated in the fire service as expressed through their various professional publications.

2.2 Concerns with Lightweight Construction

2.2.1 Firefighting Concerns

Since this project resulted from concerns expressed by the fire safety community over the performance of lightweight structural elements, a review of the specific literature that documents these concerns is helpful in order to focus on the issues.

All of the information that follows is taken directly from the referenced articles, which show the respective authors' perspectives on this topic. The intent of each article was left as is, in order to accurately show the author's point of view. Any information contained in articles known to be incorrect or misunderstood was left as originally written. Comments were paraphrased or quoted to accurately capture the author's meaning. (Note: articles in the following section are noted with a number in parentheses—e.g., '(19)', and are referenced by number at the end of this chapter. Commentary provided by the author of this report is printed in small capitals, as in, "THIS IS AN EXAMPLE OF COMMENTARY.")

2.2.2 General Concerns

Brannigan (19) clearly states that the chief interest of the fire service is the damaging force of fire, since fire destroys wooden structural members, distorts steel members, and causes connections to fail. Brannigan (30) further states that the "building [is the] firefighter's enemy."

> THESE GENERAL TENETS ARE SEEN THROUGHOUT THE FIREFIGHTING LITERATURE. IT IS CLEAR FROM THE LITERATURE THAT NOT ONLY IS THE BUILDING CONSIDERED THE ADVERSARY OF THE FIREFIGHTER, BUT SO ARE THE STRUCTURAL ELEMENTS. BY DEFINING THE BUILDING AND ITS STRUCTURAL FRAMEWORK AS AN ADVERSARY, A FIREFIGHTER CAN COME TO BELIEVE THAT THE BUILDING *IS* HARMFUL AND DEADLY—ONLY AN ANTAGONISTIC OPPONENT. THIS CONCEPT IS SEEN IN MUCH OF THE FIREFIGHTING LITERATURE ON LIGHTWEIGHT BUILDING CONSTRUCTION SYSTEMS. THE EMOTION COMES FROM THE INTENTION BY THE FIREFIGHTING LITERATURE TO ENSURE THAT FIREFIGHTERS ARE AWARE OF THE POTENTIAL DANGER, AND THAT THEY SHOULD NOT BE COMPLACENT IN LEARNING ABOUT HAZARDS IN THEIR WORKPLACE. THIS INTENT IS LAUDABLE AND NECESSARY, BUT THE MESSAGE SHOULD BE USED WITH DISCRETION, SINCE IT CAN EASILY LEAD TO MISINTERPRETATION. UNDERSTANDING ALL THE TECHNICAL ASPECTS OF THIS ISSUE IS CRUCIAL TO MAKING DECISIONS ON THE FIRE PERFORMANCE OF ENGINEERED COMPONENTS.

2.2.3 Trusses

OFTEN, THE FIRE PERFORMANCE OF TRUSSES IS CATEGORIZED GENERICALLY, THEN BROKEN DOWN FURTHER INTO THE PERFORMANCE OF WOOD, COMPOSITE WOOD AND STEEL, HEAVY TIMBER, AND STEEL TRUSSES. AT TIMES, I-JOISTS ARE DEFINED AS TRUSS CONSTRUCTION AND INCLUDED IN DISCUSSION OF TRUSS PERFORMANCE. THOUGHTS REFLECTED IN FIRE SERVICE JOURNALS ON TRUSSES FOLLOW.

2.2.3.1 General Truss Performance

Many of Brannigan's[1] (19, 23, 28) views of the fire performance of trusses can be summarized through his published statements, including:

> *A truss is a truss is a truss. Light wood, heavy timber, steel, or wood and steel combinations are equally hazardous. There are many kinds of trusses. From a construction perspective they all share the same basic advantages, which are disastrous disadvantages for firefighters...*
>
> *A truss is a minimum reserve economical structure, designed to provide a long span, that uses the least amount of material...*
>
> *A truss has no redundancy. The failure of any element of a truss entitles the entire truss to fail...*
>
> *The failure of one truss is likely to cause the failure of adjacent trusses...*
>
> *Trusses provide vast inter-connected hidden voids in which the fire can be concealed and detonated, or deflagrating carbon dioxide can accumulate...*
>
> *The collapse of trusses is sudden and catastrophic...*
>
> *Trusses collapse without warning, injuring or killing firefighters. A failure of any member can cause the failure of the truss. There is no redundancy...*
>
> *Long spans are characteristic of trusses. Failure can be catastrophic. Multiple connections characterize the truss, and all connections are vital. The failure of any connection may be fatal...*
>
> *All trusses are designed to be the lightest they can possibly be and still support the design load under normal conditions...*

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[1] For a bibliographical sketch on this author, see **Appendix B**.

> *Even if the trusses do not fail, the wide span of roof boards between them can fail...*
>
> *Steel bolts conduct heat into the wood, destroying the wood by pyrolytic decomposition...*
>
> *Multiple truss failures are the rule, not the exception, and the failure of one truss can cause serious problems to other parts of the structure, even parts away from the initial failure point...*

Mittendorf[2] (18), Dunn[2] (12), and Routley[2] (27) reinforce or expand upon some of the concepts highlighted by Brannigan above. Dunn, in particular, points out that a large area of roof deck can collapse all at once due to timber truss (A TYPE OF GIRDER TRUSS) construction.

Mittendorf (18) and Cutter[2] (36,37) recognize the fact that if a fire burns long enough and hot enough, these hazards can apply to any roof made of any structural material. These articles tend to soften the tone of the other articles by recognizing that the fire severity and duration are the real hazard to firefighters during *any* structure fire.

Finally, Brannigan (23) states that the argument is often offered that the structural elements of a truss are "protected by [a sheet of] fire-rated" gypsum. In his opinion, "this is simply not true." He reasons that the ASTM E119 test is not an accurate predictor of performance of these assemblies in a real fire. He goes on to list what he believes to be a variety of deficiencies in ASTM E119 testing, which will be detailed later in this chapter.

2.2.3.2 Wood Truss Performance

In a video entitled, "Truss Roof Collapse," Dunn (40) states:

> *There is a saying in the fire service: 'Don't trust a truss.' Why? Because a burning wood truss is the most dangerous structure you'll ever encounter when fighting a fire...*
>
> *The truss has finally been identified as the killer it is. We found out what causes burning trusses to collapse and kill firefighters, and we're passing on this information so that you can increase your chances of survival.*

He explains the concept of trusses by saying:

> *The members are joined in a succession of triangles because triangles have the required stability and strength...Unfortunately, the same triangular design makes a truss uniquely vulnerable to collapse. If one member fails, the whole truss fails, and can cause the entire roof to fail.*

The video discusses 21 firefighters who died in six truss fires. It then explains that:

> *[As can be seen], a collapsing truss roof can kill or maim by plunging firefighters on top of the roof into the fire, by burying those below the roof, or*

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[2] For a bibliographical sketch on this author, see **Appendix B**.

> *by blocking their escape from the burning building. The collapsing truss can also cause walls to collapse.*
>
> *The primary reason [truss roofs are more dangerous than solid beam construction] is the fact that a truss burns much more rapidly than a solid beam of the same dimensions, because of its high surface-to-mass ratio.*

Another reason he cites for the danger surrounding trusses is that most of the combustible material is in the truss itself:

> *It's like having a lumberyard over your head.*
>
> *[A further reason] that trusses cause catastrophe is that, unlike solid wood beams, the members are all inter-connected. Connections are always the weakest link in a building and the first parts to collapse. Trusses are full of them. The strength of the truss depends on the strength of each individual member in that section. When one chord or web fails, the entire section, even the entire roof, can fail. A truss is only as strong as its weakest member.*
>
> *[The open design of truss roofs] actually promote the spread of fire. When a blaze hits this kind of roof, flames can race through trusses unchecked. This is especially true of lightweight wood trusses because the members are so small and flimsy.*

He also comments on metal plate connectors (MPCs) used to fabricate MPC trusses:

> *Those sheet metal surface fasteners are likely to loosen—fast—whether or not the fire is hot enough to char the wood. It can cause the fastener to curl up and pull away from the truss. These* ***killer connectors*** *help make the lightweight wood truss the most dangerous of all truss roofs.*

Brannigan (17, 23, 29, 31), Peterson (1), and Routley (27) also explain that wood trusses have a much greater surface-to-mass ratio than an equivalent piece of solid wood material, saying a wood section with a greater surface-to-mass ratio will ignite sooner and burn faster than one with less. Therefore, the lightweight wood truss is a fast burner when compared to a solid wood joist or rafter. Dunn (39) further states, "...The largest combination of combustible material within the structure is found to be within the ceiling space." The wood trusses in the ceiling form a maze of 2 x 4 inch framework below the plywood roof deck. "It becomes obvious, therefore, that if a fire occurs and the building is to be saved, the fire must not be allowed to enter the roof space."

Dunn (8) and Brannigan (23, 31) both detail concerns over the open concealed space between floors. They note that there is a hazard of the void being a reservoir for explosive carbon monoxide gas, and the rapid spread of fire throughout the concealed space in all directions.

Dunn (8,39), Corbett (26), Routley (27), and Mittendorf (25) all say that there is rapid failure in wood truss assemblies. Dunn states that, according to engineering calculations and practical firefighting experience, wood trusses can be expected to collapse within approximately ten minutes in a fully developed fire. Corbett notes that witnesses at an actual fire scene estimated that the third floor collapsed only ten to fifteen minutes after the alarm was received. Routley states, "The opinion within the fire service is that wood truss assemblies increase the danger to firefighters due to structural collapse of burning structures."

Brannigan (30) points out that wood trusses are hazardous when they are extended to support balconies in apartments or commercial structures. These balconies are often an exit for occupants and access for the fire department. Fire in the truss void can impact the structural integrity of the balcony and cause the collapse of the only exit for occupants. Therefore, firefighters should not rely on the balcony or stairway as a place of refuge.

Mittendorf (25) makes a comparison between rafter assemblies and lightweight wood truss assemblies in a collapse situation. He states that the rafters and roof may collapse during a fire in the attic, but the ceiling joists will protect the firefighters below. However, this is not the case with wood trusses, since a truss assembly is the sum of its inter-connected members; therefore, if a fire is in the attic, one must expect the entire truss to collapse as a unit into the structure.

> *This was graphically demonstrated in an attic fire in California when a lightweight metal plate connected truss roof suddenly collapsed without warning into the structure, severely injuring nine firefighters.*

Dunn (8) and Brannigan (24) note that lightweight wood trusses may have defects that result in additional problems. These defects include improper storage and rough handling which cause metal connectors to pull away from the wood surface, inadequate lumber dimensions at joints with high forces, knots located in the metal connector plate contact area, gusset plates not centered on joints, gusset lugs not embedded into the lumber, defective lumber, repair of split lumber with plates, lack of fit at truss joints, inadequate connector sizes, reduced lumber sections at joints due to improper finishing, moisture in roof spaces causing rusting, and fire retardant chemicals causing corrosion of fasteners.

Brannigan (19) states that wood girder trusses often have a bottom chord consisting of four 2 x 10s side by side. The length of the chord requires splicing of the chord with gusset plates, and that results in all the splices being located at the same point. Fire at the splice point can cause the bottom chord to fail. Since the bottom chord is under tension, this failure could cause the entire girder truss to fail, dropping all the trusses attached to it.

Finally, from a different point of view, Brannigan (17) states that all the comments that are made about truss plates are not to condemn the gusset-plate truss out of hand. He notes, "Any device that conserves natural resources and reduces the cost of building certainly has intrinsic merit." From an overall fire protection point of view, the early failure of such a truss may well be beneficial in that it may open the roof and, thus, ventilate the fire. He further states that "the building will not collapse; the collapse will be a local collapse, not a general one."

2.2.4 Timber Truss Roofs

> "TIMBER" TRUSS ROOFS ARE OFTEN CATEGORIZED WITH OTHER TRUSSES IN FIREFIGHTING LITERATURE. HOWEVER, THESE TRUSSES ARE USUALLY MADE OF BIGGER PIECES OF WOOD OR TIMBERS WHICH PROVIDE LONGER SPANS. THOUGHTS REFLECTED IN FIREFIGHTING LITERATURE ON TIMBER TRUSSES FOLLOW.

Dunn (9, 13) states that the timber-truss roof is one of the most dangerous structures that exists from a firefighting point of view. It is difficult to justify a long-duration, defensive firefighting operation inside a structure with a timber truss roof. Firefighters should anticipate early collapse of the roof and subsequent failure of one of the masonry walls. However, if the timber trusses are protected by fire-retarding materials, the collapse of the roof will occur more slowly, and the timber trusses are more likely to fail one at a time. Finally, the failure of a single timber truss can cause a large section of roof to collapse due to wide on-center spacing placement.

In contrast to this, Mittendorf (18) states that his experience with timber truss roofs has led him to an opinion that does not "totally coincide with popular perception of trusses in general." The principal hazards related to truss-type roofs are said to be: weak roof, early failure rate and collapse without warning. He cites the definition of 'early' in Webster's Dictionary as being "near the beginning of a process." He then relates failure times of actual fires:

- Waldbaum's Supermarket roof collapsed 32 minutes after initial units arrived.
- the Hackensack Ford dealership roof collapsed 35 minutes after initial units arrived.
- a bow string timber truss roof that sustained a significant fire for more than 45 minutes without collapsing, while the wood-joist, flat roof in an adjacent building had collapsed.

He then notes that while timber trusses can be a very hazardous type of construction, they can also provide the strength and time needed to conduct a successful aggressive attack on fire. The key for personnel is to have a working knowledge of the hazards of timber trusses and adhere to the appropriate on-site fire size-up criteria.

2.2.5 Connections

2.2.5.1 Truss Plate Connectors

Brannigan (17), Dunn (8), Routley (27), and Peterson (1) all point out that the "sheet metal surface fastener" is a major concern of the fire service. This is due to the feeling that the fastener collects heat and transmits it through the prongs, destroying wood fibers along these prongs by pyrolytic decomposition. Once this decomposition takes place, the entire wood truss fails.

Dunn (8,12,40) states further that these surface fasteners are "a deficient structural connection from a fire protection point of view"—and—"a dangerous structural connection." He also states, "...The heat from a fire can warp the thin sheet metal surface fastener, causing it to curl up and pull away from the wood truss." Therefore, from a fire protection point of view, "The sheet metal surface fastener is an inferior, dangerous type of connector, because the connection points are the first to fail." Finally, he calls these fasteners "killer connectors."

Manny (20) and Brannigan (24) also suggest that the argument that metal plate connectors act as heat reflectors has not been studied thoroughly enough to demonstrate that this has any bearing on the performance of a connector in a real fire condition.

Brannigan (23), Mittendorf (21), and Dunn (12, 39) all note that metal gusset plates, sheet metal surface fasteners or gang nails may be a problem in a fire. These fasteners may be effective truss connectors when tested in a laboratory, but from a fire protection point of view, they are deficient. They state that as the gusset plate heats up, it conducts heat to the prongs, or v-shaped points, which will cause the wood to expand. The wood is then destroyed by pyrolysis, which causes the gusset plate to fall out. Since the prongs are only 3/8 in. to 1/2 in. long [DEPENDING ON THE AUTHOR OF THE ARTICLE], the metal connector will not last very long under fire conditions. (SEE RELATED DISCUSSION ON PAGE 35 UNDER **Industry Literature**.)

2.2.5.2 General Connections

Manny (20) suggests that over time, connectors have a tendency to work their way out of structural members, and need reseating. This is due to drying and shrinking of wood, the settling of buildings, and vibrations from people, machinery, nearby traffic, etc. Therefore, the fire service should expect less stability and an earlier failure potential of lightweight wood truss assemblies as structures age.

Finally, Dunn (8) and Brannigan (23) note that fire vulnerability of connections due to fire is often overlooked. In any structural element, the point of connection may be the critical area subject to a failure during a fire. When one connection fails, it allows the entire system to fail. **(**SEE RELATED DISCUSSION ON PAGE 35 UNDER **Industry Literature.)**

2.2.6 Wooden I-Joists

> I-JOISTS ARE A RELATIVELY NEW KIND OF LIGHTWEIGHT ENGINEERED BUILDING COMPONENT. THOUGHTS REFLECTED IN FIRE SAFETY LITERATURE ON I-JOISTS FOLLOW.

Whitfield (4), Brannigan (12) and Clark (32) all state that the flames in a fire will quickly penetrate the thin web members of I-joists. Brannigan (12, 34) and Clark (32) further state that the surplus wood that makes it possible for firefighters to stand and operate on a burning structure is no longer available in wooden I-joists. This is because the web is thin and has holes cut into it to accommodate utilities. Once the fire reaches the I-joist, the plywood burns at a high rate of heat release, and fire extends through the holes, so that both sides of the joist burn rapidly. As soon as the plywood starts to burn, the I-joist loses strength. There is no reserve—there is no margin for safety. Again, expect early collapse.

Clark (32) notes that bonding adhesives have a flammable base. Ignition can be expected at relatively low temperatures, accelerating system failure. Fire causes the I-joist system to revert to its individual components due to adhesive bond degradation. This usually leads to sudden structural collapse. Therefore, Clark (32) states that members may lose strength in five minutes or less without providing warning.

2.2.7 Wood Joist Performance

> WOOD JOIST CONSTRUCTION IS OFTEN VIEWED AS THE BASELINE OF COMPARISON WHEN EVALUATING THE FIRE PERFORMANCE OF STRUCTURAL ASSEMBLIES. THOUGHTS REFLECTED IN FIREFIGHTING LITERATURE ON WOOD JOIST CONSTRUCTION FOLLOW.

Brannigan (28) states that the fire service should not put total emphasis on truss hazards, as this may lead to the erroneous conclusion that sawn joist or rafter roofs are completely safe. They simply have different defects.

Routley (27) and Dunn (7) note that a protected joist assembly seldom fails catastrophically. This is due to joists providing a built-in fire stop, avoiding rapid involvement of the entire void space.

Routley (27) and Schaffer (35) note that solid-sawn lumber components are said to provide warning of imminent collapse by gradually sagging under the fire load. Routley goes on to say that failure of one joist is seldom catastrophic because the remaining joists have more resistance to load transfer than trusses do. This is because of the increased chance that adjacent trusses are approaching their own point of failure when the initial truss burns through, which is not the case with joist construction.

Routley (27) states that while the fire endurance rating for unprotected joist assemblies is similar to that for unprotected truss assemblies, they do not have the same reputation for sudden collapse. Routley also notes that trusses often span wider spaces than joist systems, and joists are often supported by a partition wall system below. This makes joist systems safer for firefighters.

Dunn (7) notes that wood joist systems collapse in three different ways when attacked by fire: 1) the wood deck may burn through and collapse; 2) several floor joists may fail, causing a localized failure of the floor; and 3) a large section or entire floor level fails, sometimes causing failure of adjacent walls or floors below.

In the same article, Dunn notes that the collapse of wooden joist support systems does not occur as readily in residential buildings. The reasons for this are: 1) floors in residences are usually not as heavily loaded as floors in commercial buildings, 2) floors in residential buildings are subject to fewer structural alterations than those in commercial buildings, and 3) the underside of a floor in a residence is often protected from fire by a ceiling. Dunn also states that one should be most concerned about the bathroom floor in a residence, as it collapses more often than the other floors, due to the plumbing penetrations in a bathroom floor and the potential for rotting due to moisture.

Finally, Brannigan (17) notes that the "surplus wood of a sawn wooden beam makes it safer for firefighters to stand and operate on the burning structure." As long as only the fat is burning, the firefighter is relatively safe.

2.2.8 Steel Performance

FIREFIGHTING LITERATURE ALSO STATES THAT EVEN THOUGH STEEL IS NON-COMBUSTIBLE, IT HAS ATTRIBUTES THAT CAUSE PROBLEMS IN FIRES. THOUGHTS REFLECTED IN FIREFIGHTING LITERATURE ON STEEL FOLLOW.

2.2.8.1 General Steel Performance

Dunn (11) and Brannigan (16) relate that the failure temperature of steel is near 1000° F. At this temperature, the steel will lose 40% of its load-carrying capacity, and exert its greatest thrust due to expansion. Brannigan (16, 17, 34) notes that the coefficient of expansion of steel is such that substantial elongation can take place at ordinary fire temperatures. Elongating steel has been known to push down walls that are far from the location of the fire. Personnel on the roof a good distance from the fire area have been caught in the collapse.

Brannigan (17), Sylvia (38) and Dunn (11) note that steel is non-combustible, and leads to unwarranted confidence in its fire proof capabilities and suitability for all applications where fire is a problem. Unprotected steel has no fire resistance, and, consequently, a steel building can be destroyed by fire. The building itself will not burn, but it is likely to collapse during an interior fire due to burning contents. When a working fire occurs in a non-combustible building, firefighters must expect sections of the building to collapse.

Brannigan (33) states that steel girders are being used with increasing frequency as main structural elements. Building officials apparently believe the gypsum sheathing from the floor/ceiling assembly also protects the steel. "This is unevaluated." Should the steel be exposed to the fire in the concealed space, the steel will elongate, and, if restrained, will rotate on its axis and overturn, dumping all the trusses on it. This would cause a sudden collapse of a large section of the building.

Finally, Dunn (11) states that heated steel will bend, sag, warp, and twist unless it is covered, encased, or enclosed with some type of insulating material.

2.2.8.2 Steel Bar Joists

Brannigan (16, 34) and Dunn (10, 11, 12) state that steel bar joists may collapse after five to ten minutes of exposure to fire. Brannigan further states that when bar joists were tested using the ASTM E119 standard test for fire resistance, "they failed within seven minutes.

This failure rate can be compared with two-in. thick wooden beams,[3] which lasted only ten minutes."

Brannigan (16) relates that in tests done at Underwriters Laboratories, bar joists 30 ft. above a light-combustibles fire reached 1540° F in a little over five minutes. At this temperature, the steel bar joists would begin to rapidly lose their strength.

Dunn (11) and Brannigan (16) express concern about the spacing of steel bar joists. A wide on-center spacing (e.g., six feet on center) is not unusual for steel bar joists in a roof. Firefighters cutting a vent opening may find themselves standing on the cantilevered end of a corrugated steel sheet for only a very short period time, because the steel roofing will not support their weight.

Brannigan (16) and Dunn (12) state that unprotected steel bar joist trusses are particularly dangerous in a fire. This is due to steel being such a strong material that pieces with a very small cross-section can be assembled into trusses to provide long clear spans. Trusses are also inherently unstable, and therefore need to be tied together to resist overturning. These ties or braces transmit undesigned torsional loads from one truss to another, resulting in multiple truss failures during a fire.

Finally, Brannigan (30) notes a particular fire situation where unprotected bar joists formed the basement floor in a commercial building in Rockville, Maryland. There, a basement fire caused a joist to fail and the floor to open before employees on the first floor could get out of the building.

2.3 Other Related Products

2.3.1 Glue-Laminated Beams

> GLUE-LAMINATED BEAMS ARE CLASSIFIED AS HEAVY TIMBER CONSTRUCTION WITHIN THE BUILDING CODE. THOUGHTS REFLECTED IN FIREFIGHTING LITERATIRE ON GLULAM BEAMS FOLLOW.

Clark (32) states that glue-laminated wood beams can be as dangerous as any truss and should be treated accordingly. His reasoning is that under fire conditions, the strength of a glue-laminated beam deteriorates rapidly as it reverts back to its individual components. This belief is centered on the fact that the bonding adhesives used to manufacture glue-laminated beams have a flammable base. Ignition of these adhesives can be expected at relatively low temperatures, accelerating system failure. Finally, he states that in structures with laminated beams, one must expect early collapse.

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[3] These are assumed to be the same as '2 x' wood joists.

2.3.2 Heavy Timber Construction

> THIS CONSTRUCTION TYPE HAS A SEPARATE CLASSIFICATION IN THE MODEL BUILDING CODES, AND IS KNOWN FOR ITS FIRE RESISTIVE PROPERTIES. THOUGHTS REFLECTED IN FIREFIGHTING LITERATURE ON HEAVY TIMBER CONSTRUCTION FOLLOW.

Dunn (10) states that heavy timber construction does not collapse during the early stages of a fire. Masonry walls, large timber girders and columns characteristic of this construction are very stable during the growth period of a fire. The problem with heavy timber construction is the radiating heat which will not allow firefighters to get close to the building, and which spreads to adjacent buildings. Once timbers become engulfed in fire, control of the structure fire is impossible. After several hours, the floors will collapse and free-standing walls will fall into the street.

2.4 Other Related Concerns

2.4.1 Concealed Spaces

Brannigan (30) notes the temptation to use sizable concealed spaces in attics for storage, maintenance shops, etc., and that a fire entering this truss void can be very dangerous. He recommends that sprinklers be required throughout concealed spaces. Dunn (39) adds that since the model building codes require fire stopping after only 3,000 $ft.^2$ of area is built, many structures will not have firestopping divisions. If fire enters the concealed area, it will spread and involve the entire cockloft. A roof space containing a truss system will allow fire to spread more quickly than one containing solid wood joists. Fire spreads between the trusses and open web members more rapidly.

2.4.2 Testing For Fire Performance

Building codes generally permit the use of structural assemblies based upon performance in standardized fire tests such as ASTM E119. This testing has also been the subject of comment.

Brannigan (24), Dunn (10) and Routley (27) all note that firefighters cannot use this fire resistive test to estimate the structural stability of a burning building. They point out that the question firefighters have is not the test results, but the actual performance of all components under *real* fire conditions. ASTM E119 provides a theoretical basis for comparison, but it does not reproduce conditions encountered in real building fires.

Dunn (10) and Brannigan (24) both relate that the floor may collapse even though it has a 1-hour fire resistance rating, because:

- The actual fire may be more intense than the test fire.
- A small-scale sample of a floor cannot be used to predict the way a full-scale floor will act in a fire.

- The bearing walls, columns, or girders supporting the floor may collapse.
- The fire may burn undetected for longer than the 1-hour test period.
- The workmanship and materials of the actual building may be inferior to those of the test.
- The test does not simulate fire entering the truss void laterally.
- The test does not provide for any penetrations of the gypsum sheathing.
- The test does not provide for additional air being added to the assembly through deficient fire stopping.
- The test does not provide for a moving live load with some impact component, representing firefighters making a primary search for victims.

Routley (27), Brannigan (24, 30) and Cutter (36) note the concern that actual construction is seldom built like the tested assembly. The test assemblies are often built by the organization sponsoring the test so, as a consequence, the construction is perfect in every detail. They point out that buildings are rarely built perfectly, and that building inspectors cannot ensure that the exact assembly specifications are met in the field.

Finally, Brannigan (24) notes that the ASTM E119 time/temperature curve does not reflect real fire conditions. The National Bureau of Standards (NBS) performed a study and recommended a curve that more accurately reflects actual time/temperature conditions during a fire. He recommends that this time/temperature curve should be used.

(See related discussion on page 35 under **Industry Literature**.)

2.4.3 Building Code Concerns

The literature contains comments expressed in firefighting literature on the adequacy of building codes. These concerns can be summarized as follows:

Brannigan (34), Peterson (1) and Ryan (3) voice concern that firefighter safety has never really been addressed in building code regulations. The responsibility of firefighter safety is left to the fire department.

Brannigan (34) and Ryan (3) both acknowledge that building code requirements are not self-enforcing, and therefore regulations regarding draftstopping, firestopping, and compartmentation can be violated.

Brannigan (16) notes that most buildings have no protection from fire attack, since these buildings are legally classified as non-fire-resistant. It is acceptable—even expected—that a building will collapse in a fire. In a building code, as long as the structure carries its normal load, it is irrelevant that one type of building will collapse faster in a fire than another. Even buildings classified as non-combustible are subject to early failure in a fire.

Finally, Brannigan (34) states that in a combustible structure[4] involved in a fire, no code provision—however well-written, however well-meaning—provides personal safety for the firefighter. "The building is the enemy, and we must know the enemy."

2.4.4 Collapse Experience

> THE PRIMARY REASON FOR THIS STUDY IS COLLAPSE OF LIGHTWEIGHT BUILDING COMPONENTS WHICH HAVE CAUSED FATALITIES. SOME KEY INCIDENTS THAT HAVE HIGHLIGHTED CONCERN FOLLOW.

Brannigan (17) notes that six firefighters were killed in New York when a bow-string truss collapsed during a supermarket fire. Time from initial alarm to initial collapse was 37 minutes. The entire roof collapsed 25 minutes later. A Tempe, Arizona, firefighter was killed and several others narrowly escaped death when a wood truss roof[5] on a one-story restaurant collapsed 14 minutes after arrival. A fire in Ottawa, Kansas, for which a pre-fire plan existed, showed that the roof was supported with open, unprotected metal trusses, and that the firefighters should anticipate rapid roof collapse. The rear half of the roof collapsed 11 minutes after arrival of the fire department, and the remainder collapsed 10 minutes later.

Mittendorf (21) states that several structure fires in Southern California have graphically illustrated the partial or total structural failure of lightweight construction in a short time. In December of 1979, the Orange County Fire Department responded to a structure fire in a single-family dwelling with fire showing from the garage. Approximately 10 minutes after arrival the entire roof collapsed, injuring nine firefighters. During August 1981, the Los Angeles Fire Department responded to another common structure fire. First-in companies observed a one-story, multi-occupancy commercial building with a small amount of fire showing from the roof over the involved occupancy. Approximately 2 to 3 minutes after the arrival of the initial companies, the entire roof over the involved occupancy collapsed. The roof was of open-web construction.[6]

Dunn (7) notes that on October 17, 1966, 12 firefighters were killed when the joist floor[7] of a Wonder Drug Store in New York City collapsed. The first floor collapsed suddenly into the cellar without any warning signs, hurling 10 firefighters, company officers, and chiefs into the burning cellar. On June 17, 1972, in Boston, nine firefighters were crushed to death when the solid-sawn joist floors of the Vendome Hotel suddenly collapsed. Before these structural failures, there were no warning signs. Firefighters had no time to act and withdraw to safety, and no satisfactory explanation of the incidents followed.

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[4] All buildings are combustible to some degree.

[5] The specific type of truss was not stated in the article.

[6] The specific type of open web construction was not stated.

[7] It was assumed to be solid sawn wood joist.

However, a later report by the NFPA (41) on the Vendome fire provides a satisfactory explanation of the incident. The report states that:

> *...The collapse began when the seven-inch cast iron column lost its support. This was caused by a failure of the masonry bearing wall under the column. Failure of the column caused failure of the masonry wall supporting...the third, fourth and fifth stories and the roof, and as this wall dropped, floor joists were pulled from wall pockets...*
>
> *The renovations had caused excessive stresses on the bearing wall under the cast iron column. Only a small additional stress increase was required to cause failure...*

Dunn (40) headlines the following cities where fatal truss collapses have occurred:

- Orange County, Florida: two firefighters die when a lightweight wood truss roof caves in during a store fire.
- Hackensack, New Jersey: a bowstring timber truss roof collapses in garage fire—five firefighters are dead.
- Irving, Texas: A firefighter dies when the lightweight wood truss roof of a condominium collapses.
- Brooklyn, New York: a bowstring timber truss roof collapses in supermarket blaze—six firefighters are killed.
- Cliffside Park, New Jersey: five firefighters are killed in fiery collapse of bowstring timber truss roof.

In the Orange County, Florida, fire reported by Dunn above, a fire report by Edwin J. Spahn (43) states the following about the performance of wood truss assemblies:

> *The wood truss assemblies performed, generally speaking, as expected. There is evidence that the fire condition had proceeded through the initial development, 180 seconds to 300 seconds and into free burn, prior to discovery of the fire. There is evidence and statement to lend credence to the proposition that the fire entered into its initial development stages shortly before 1530 hours. Using these estimates, it is probable that more than one truss assembly and attendant top chord stabilizing roof sheath*[8] *had been subjected to continuing fire damage for approximately 25 to 30 minutes before collapse.*
>
> *The ultimate collapse of the truss and roof assemblies were accelerated because of several factors not provided for in current codes and standards.*

Routley (42) reports that four firefighters died on December 20, 1991, in Brackenridge, Pennsylvania, when a lightweight concrete floor supported by unprotected lightweight steel

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[8] Plywood sheathing is assumed.

joists collapsed into the basement of a two-story building. The construction was of 1920 vintage.

Peterson (1) describes a single-family dwelling that used floor trusses throughout the floor system. When firefighters arrived at the scene, there was a total loss of the upper floor hallway between the bedrooms and the living room. There was also a definite loss of integrity of the support structure for the upper-level floor and the interior walls. It became apparent from the analysis of this fire that a dangerous fire safety problem existed with this type of construction, and a solution to the problem had to be found.

2.4.5 Warning Signals

> As expressed in the foregoing comments, the concerns with construction focus on unexpected collapse or collapse without warning by the structural elements. Additional generalized concerns regarding this issue are summarized as follows:

Brannigan (30, 34) notes that firefighters cannot rely on outdated concepts, such as: floors will sag, floors and roofs soften, water will flow through bricks, smoke will push out of mortar joints, and strange noises will take place before collapse occurs. If firefighters rely solely on these warning signs for indication of collapse in today's lighter buildings, disaster will be the certain result.

Dunn (7) suggests that the floor deck of rooms with no ceilings below them will fail before the floor joists are weakened. Any time the floor deck appears spongy or weakened during a fire, the floor below must be examined for fire.

Routley (27) states that Schaffer's analysis (35) assumes that the warning signs will occur as predicted, and will be observed by someone who is in the right place to recognize the danger soon enough to warn everyone else. Routley and Brannigan (28) go on to state that the warning signs may be present and not recognized, or not present at all, and thus, do not allow enough time to escape.

Finally, Corbett (26) notes that in a San Antonio fire, firefighters operating on the third floor noted that prior to collapse, there was no flame-through of the flooring above the truss, and no sagging of the floor. The only indication of a problem with the floor was its feel of "sponginess". The potential for immediate collapse was indicated by the fact that the fire was burning through the exterior veneer at the location of the floor trusses, which meant the truss' concealed space was fully involved. A company officer decided to evacuate the area, and approximately 35 to 40 seconds after sponginess was indicated, the third floor collapsed. It was estimated that the third floor collapsed 10 to 15 minutes after receipt of the alarm at the fire alarm office. It appears that the fire was reported soon after ignition took place.

2.4.6 Tactical Issues

> The fire safety community recognizes that firefighting tactics are constantly revised with the advent of new construction processes.

THOUGHTS REFLECTED IN FIREFIGHTING LITERATURE ON FIREFIGHTING TACTICS FOLLOW:

Brannigan (17, 23, 31) and Dunn (11) note that it is extremely dangerous to apply tactical operations based on experience with sawn beams[9] to other structural members such as trusses, wooden I-beams, and steel bar joists. It is no longer wise to employ tactics which have the assumption that firefighters are working on wood joists.

Dunn (8), Cutter (37), Brannigan (17, 19, 34), Mittendorf (21), Routley (27), and **Engineering News Record** (5) all suggest that there is no substitute for the fire department developing a system of accumulating and organizing information for pre-fire planning, and then performing follow-up inspections at the building site to further refine this plan. This information should be used in the development of suppression tactics for the building and standard operating procedures based on the collapse potential of that building.[10]

Cutter (36), Brannigan (28) and Dunn (11) all express concern over the ventilation procedures used by fire departments. Where ventilation is necessary, consideration should be given to horizontal ventilation, working from a roof ladder, or working from a ladder truck to accomplish the ventilation. Having steel bar joists, wood trusses, or wood I-joists in a roof system creates a hazard to the firefighter on that roof, if the firefighter is expecting solid-sawn joist construction and uses typical venting procedures.

Brannigan (17, 34) suggests that all firefighters should be aware of whether the fire is a structure fire or a contents fire. Once the fire becomes a structure fire, firefighters do not belong on the roof, or on or beneath the floors. It should be announced immediately that it is a structure fire and all personnel should evacuate the building immediately.[11]

Brannigan (34) and Sylvia (38) note that a safe rule for steel trusses is that a fully involved, non-combustible building is about to collapse. A lightweight steel truss building has almost no inherent fire resistance. If there is enough fire to justify a second alarm, it is almost a certainty that the building is unsafe to enter.

Dunn (6) suggests the following tactics to help firefighters prevent injury or death:

- When operating at a private home with a sloping roof, it is probably more effective to vent the top floor windows than to cut a roof vent.
- The pre-fire plan should include the type of roof construction.
- The fire department should develop standard procedures for operating on sloped roofs. This should be based on life safety, fire containment, and property protection—in this order of importance.

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[9] E.g., solid-sawn lumber joists and rafters, like 2 x 10s.

[10] This is the single-most mentioned activity that could reduce the risk to firefighters in fighting structure fires.

[11] It is assumed Brannigan implies a fire with the structural elements involved.

- Sloping roofs are generally designed to support less live load than a flat roof. Therefore, sloping roofs will support fewer firefighters.
- Firefighters should understand that when they walk on the roof of a burning building, they risk the possibility of falling through and not being able to get out alive.

Dunn (7) further suggests that to avoid plunging through a burning floor deck, encroaching firefighters should keep one leg outstretched in front of them as they move forward. Another technique that can be used is to drop an axe or a halligan tool onto the floor in front of the firefighter before advancing. These techniques will not protect a firefighter from a floor joist collapse, but they will indicate a weakened floor deck above the floor joist.

Finally, the Illinois Fire Service Institute (22) suggests these tactics:

- Pre-plan all new construction and any remodeling that use lightweight components.
- Modify fire department tactics to open concealed spaces quickly.
- Maintain records of all buildings that uses lightweight components.
- Be aware of the time factor—always ask, "How long has the fire been burning?"
- Remember, some floor systems give no warning prior to collapse.

2.4.7 Fire Safety

Peterson (1) states that fire safety relates to the following areas of concern: life safety, property protection, and continuity of operations. He notes that the degree of risk that will be accepted by occupants is a difficult decision, at best. In a residential home, he suggests that the level of protection equivalent to the standard 2 x 10 floor joist construction would be desired for any fire safe design using a wood truss floor or other lightweight construction material. The solution he suggests for lightweight construction is to use a drywall ceiling in all unprotected areas. This would allow adequate escape time for occupants and firefighters, and sufficient protection for firefighters from unexpected collapse.

2.5 Concluding Remarks Found in the Literature

> MANY OF THE ARTICLES FOUND IN THE LITERATURE DREW CONCLUSIONS WITH RESPECT TO FIRE PERFORMANCE, AND AT TIMES RECOMMENDED ACTION FOR ADDRESSING THE PERFORMANCE CONCERNS. THESE CONCLUDING REMARKS ARE SUMMARIZED HERE.
>
> GENERALLY, MUCH OF THE FIREFIGHTING LITERATURE RECOGNIZES THE NEED FORE CONSTANTLY UPDATED, DETAILED PRE-FIRE PLANNING, AS WELL AS THE NEED FOR EARLY DETECTION AND SUPPRESSION (E.G., SPRINKLER) SYSTEMS, EXPANDED FIRE PREVENTION EDUCATIONAL EFFORTS, AND THE MOST INFORMED FIREFIGHTING TACTICS.

Brannigan (24) and Peterson (1) note that in reference to trusses, any device that conserves natural resources or reduces the cost of buildings has intrinsic merit.

Mittendorf (18) reminds readers that by focusing on roof truss hazards, one may be distracted from remembering and recognizing that all roofs can pose serious dangers. Any roof can be fatal if the proper ratios of fire, time, and construction type are correctly combined. The amount of time before failure cannot be predicted for a roof of any type. However, any roof can be dangerous and collapse unpredictably during the early stages of a fire.

Summers (4), Manny (20), Cutter (36), and Grundahl (2) all suggest that a pro-active, cooperative approach involving both sides is needed to learn more about the actual performance of these components in fires. This is information and feedback that firefighters need to better predict field performance. Straesesky and Weber (22) state that there has been considerable speculation concerning what floor systems might do under fire conditions, but that there has been very little information published on this subject. Schaffer (35) and Mittendorf (21) suggest that the focus should be on the time required for failure, the speed or rate of failure, and warning signals.

Brannigan (24) cautions against accepting the claims of the lightweight component industry, based on obsolete test procedures.

Cutter (37) and Routley (27) recognize that the economics of wood truss construction[12] are a reality, and that wood construction is here to stay. Routley (27) goes on to state:

> *The fire service must recognize the dangers presented by these construction methods and concentrate on pre-fire planning and fire ground safety to reduce the risks.*

Finally, Manny (20) and Cutter (36) suggest that manufacturers and fire service personnel need to work together to define appropriate actions to protect lives, since neither the fire service nor the lightweight building component industry are going to go away.

2.6 Industry Literature

2.6.1 Truss Plate Connectors

> INFORMATION ON THE PERFORMANCE OF TRUSS PLATE CONNECTORS WAS PROVIDED TO THIS PROJECT IN THE FORM OF PHOTOGRAPHS AND VIDEOTAPES. DETAILS FOLLOW.

In a series of photographs taken by Haan (14), the performance of connector plates is shown after trusses have been involved in a fire in a residence (See Figures 1a-c). These photographs show that the truss plates do protect the wood beneath the truss plate from the heat of fire by reflecting the radiant energy. This was also said to be the case in an article by Schaffer (35).

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[12] And more broadly, lightweight building components

The photographs also show that the truss plates appear to have pulled away somewhat from the wood after involvement in the fire. There is no clear reason for the gaps to be present in the plated joints. The cause of the plates pulling away from the wood at the joints as they have in these photographs is unknown; one possibility is some combination of expansion of the steel and shrinkage of the wood.

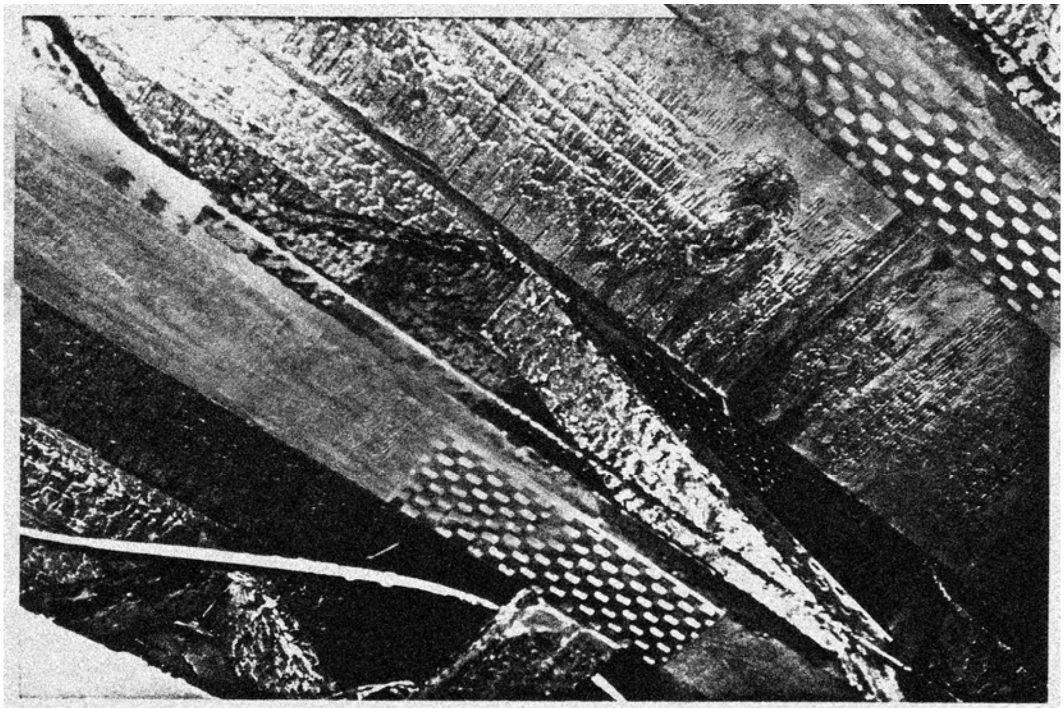

Figure 1a. Photograph of a bottom chord splice plate (14).

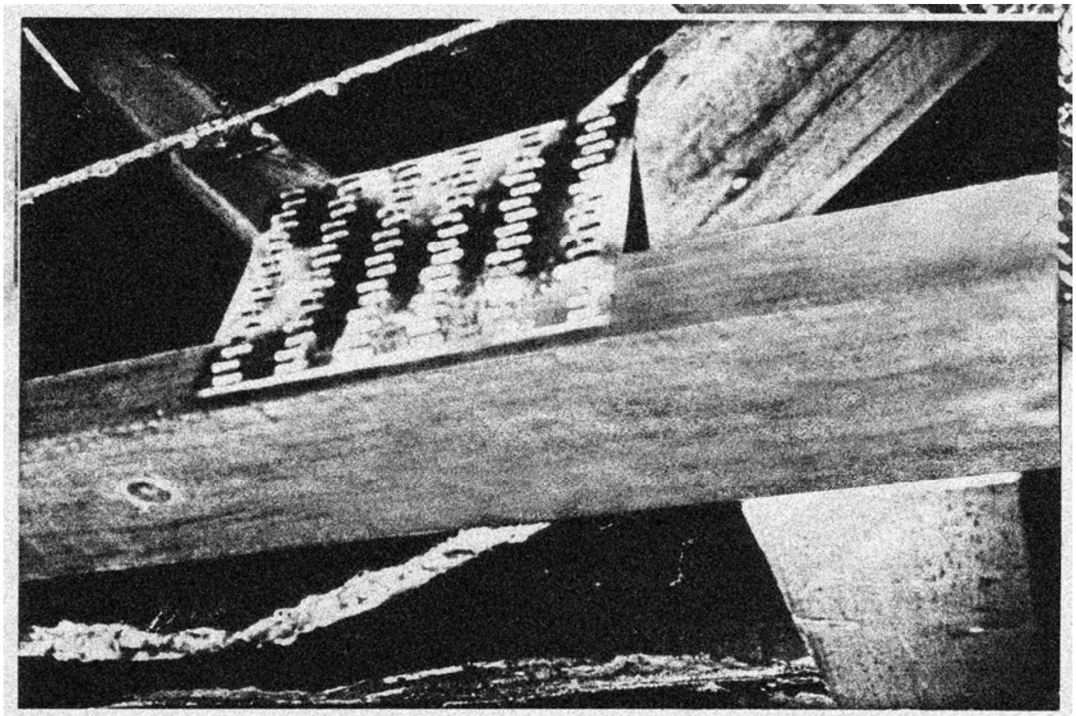

Figure 1b. Photograph of a bottom chord joint connection (14).

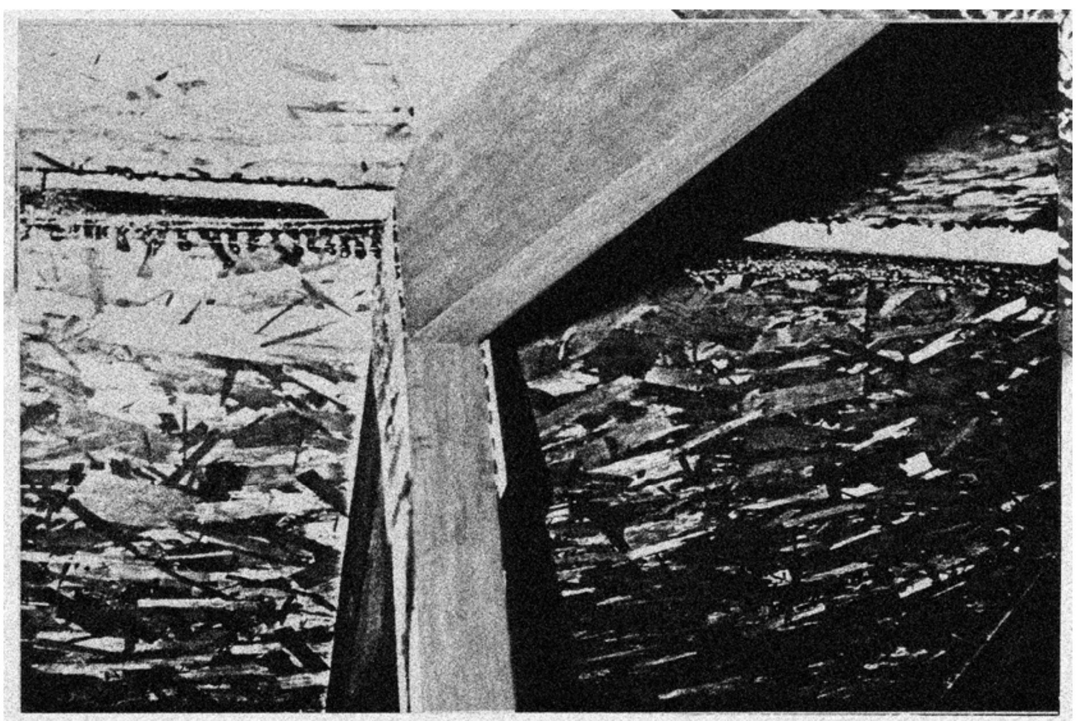

Figure 1c. Photograph of a peak joint connection (14).

A Weyerhaeuser Fire Technology Laboratory videotape (15) of a test on a truss plate splice joint under ASTM E119 fire exposure conditions shows a specimen as it goes through the various stages of fire endurance performance. This includes a reflectivity phase, where the wood is being protected by the truss plate; a conduction phase, where the wood underneath the truss plate becomes charred; and a failure phase due to the charring of the wood below the plate. The failure of the plate is due to the applied load on the truss, not by the truss plate's curling up or warping away from the wood due to fire exposure.

Schaffer (35) states that in the United Kingdom and Australia, lumber spliced with connector plates have been fire tested. Under conditions of full tensile design load and full fire severity, the times to failure were 4 minutes in the Australian test and 6.5 minutes in the British test.

2.6.2 General Connections

Schaffer (35) notes that testing has been done on continuous lengths of 2 x 4 lumber under full design load and fire exposure. Average times to failure for the solid 2 x 4 were 9.5 minutes for Coast Douglas Fir, 11.7 minutes for Southern Pine and 12 minutes for Messmate. The Messmate was tested with a finger joint, and its average time to failure was 9 minutes, 75% of the time to failure for the continuous lumber.

2.6.3 Testing for Fire Performance

> THE ASTM E119 TEST HAS COME UNDER SCRUTINY AS A SUITABLE TEST METHOD. COMMENTS ON ASTM E119 IN THE GENERAL LITERATURE FOLLOW.

Ryan (3) notes that the ASTM E119 scope statement, which is found in most fire test standards, contains the following caveat:

> *This standard should be used to measure and describe the properties of materials, products, or assemblies in response to heat and flame under controlled laboratory conditions and should not be used to describe or appraise the fire hazard or fire risk of materials, products, or assemblies under actual fire conditions. However, results of this test may be used as elements of a fire risk assessment that takes into account all pertinent factors of the fire hazard of a particular end use.*
>
> *The results of these tests are one factor in assessing fire performance of building construction and assemblies. These methods prescribe a standard fire exposure for comparing the performance of building construction assemblies. Application of these test results to predict the performance of actual building construction requires careful evaluation of test conditions.*[13]

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[13] ASTM E119-83 **Standard Methods of Fire Tests of Building Construction and Materials**.

Ryan (3) states that an ASTM E119 1-hour rated assembly is expected to collapse just after the 60 minute period of exposure. This should be recognized by everyone dealing with ASTM testing and dealt with accordingly.

Finally, Ryan (3) points out that research has shown that the ASTM E119 time/temperature curve does not follow curves developed in other fire scenarios. Some fires exceed 1700° F in early stages. Other scenarios show that the temperature never exceeds 1200° F for long periods of time. He queries, "Which is more hazardous: the fast-growing fire that drops off, or the steady, slow-burning, temperature-increasing, long-duration fire?" Further, he says that it is impossible to test assemblies under all possible conditions. The E119 time/temperature curve has been judged by knowledgeable experts to best represent a relatively severe-intensity fire for use in a comparative assessment of the adequacy of assemblies for protecting building occupants, and the spread of fire in compartments.

2.7 Summary of Concerns with Lightweight Construction

2.7.1 Firefighting Concerns

A SUMMARY OF THE ISSUES THAT WERE EXPRESSED IN THE FIREFIGHTING LITERATURE FOLLOWS. CONTENT HAS BEEN LEFT AS STATED IN THE ARTICLES. DISCUSSION OF THIS CONTENT CAN BE FOUND IN **CHAPTER 7: DISCUSSION**.

2.7.1.1 Trusses

The first major concern regarding a truss is that if one element of a truss fails, the entire truss fails. This suggests that there is no redundancy within a truss, and that it resembles a series of pin-end connected members. This concern is taken one step further in that the failure of a single truss will also cause failures of adjacent trusses. Trusses are said to collapse without warning, injuring or killing firefighters. Multiple truss failures are said to be the rule rather than the exception, and the failure of one truss causes serious problems to other parts of the building.

Trusses are also designed to span very long distances using the smallest amount of material possible. The triangular configuration of trusses creates a concealed space that is open to the passage of flames and hot gasses throughout the floor/roof cavity. This allows for the potential rapid extension of the fire to other areas of the building.

Finally, trusses are said to consist of multiple connections that are all vital to performance. The failure of any connection may have fatal consequences.

2.7.1.1.1 Wood Trusses

Wood trusses have all of the performance characteristics of trusses as defined above. Their performance is also characterized more specifically. It is noted that wood trusses have a greater surface-to-mass ratio than joist construction, so that they will ignite sooner and burn faster. It is also believed that a great amount of combustible material lies within the concealed space of a wood truss assembly. Wood trusses are expected to collapse within approximately 10 minutes in a fully developed fire.

It is pointed out that trusses are often extended to support balconies in apartments and commercial structures. Fires in trusses using this construction style can cause the collapse of the only exit for occupants and firefighters. Therefore, firefighters should not rely on the balcony or stairway as a place of refuge.

It is also noted that wood trusses may have defects that cause structural problems. These defects normally occur during the manufacturing process, and may contribute to the early collapse of a truss during a fire. Finally, literature and educational videos identify the wood truss as a killer, and state that it is the most dangerous structure that exists from a firefighting perspective.

2.7.1.1.2 Truss Plate Connectors

In the literature and educational videos, connections are viewed as a deficient structural connection from a fire protection point of view, and are referred to as "killer connectors." They are dangerous because the heat from a fire can warp the thin sheet metal surface fastener, causing it to curl up and pull away from the wood truss. The metal surface fastener also conducts heat, causing wood fibers adjacent to the teeth to be destroyed by pyrolytic decomposition. Once the wood is destroyed at the connection, the entire truss fails. These fasteners may be effective truss connectors when tested in a laboratory for structural strength; but when they are subject to fire, they are deficient.

Finally, there is concern within the fire service that vibration from normal building activities may cause the truss plates to loosen over time. This could contribute to the early failure of truss plate connections in a fire. The point of connection is the critical area subject to failure during a fire. When a connection fails, it allows the entire system to fail.

2.7.1.1.3 Timber Truss Roofs

Timber truss roofs are claimed to be one of the most dangerous structures because of early collapse of the roof and potential failure of the masonry walls. Also, timber truss roofs are often built with wide on-center spacings, causing large sections of the roof to collapse if the truss collapses.

Conversely, there is the view that timber truss roofs, although hazardous, can provide the strength and time needed to conduct a successful aggressive attack of the fire. The key to fighting these fires is to adhere to appropriate on-site fire size-up criteria, and to possess a knowledge of timber truss roofs.

2.7.1.2 Wooden I-Joists

A concern surrounding wooden I-joists is that the adhesive used for bonding may deteriorate during a fire, causing the I-joist to fall apart. Another concern is that the web material is very thin and often has holes cut into it. This allows the fire to extend through the holes, and burn both sides of the joist rapidly. It also allows for the extension of the fire.

2.7.1.3 Wood Joist Construction

It is stated that one should be aware that sawn joists and rafter roofs are not completely safe—they simply have different characteristics. It is noted that joist assemblies seldom fail catastrophically. Should one joist fail, the others will support the existing load. A concern in joist construction is the bathroom area in the residence. Floors in the bathroom collapse more often than those in other areas due to plumbing penetrations and rotting due to moisture. Finally, it is stated that joists are safer for supporting the weight of firefighters in a burning structure. As long as the "fat" of the joist is burning, the firefighter is relatively safe standing on the floor or roof.

2.7.1.4 Steel Member Performance

Steel is known to expand dramatically and lose approximately one-half of its load-carrying capacity when temperatures near 1000° F. Expanding steel often causes problems with other structural elements in a fire. Steel is non-combustible, which often leads to unwarranted confidence in its fire-resistive properties. Unprotected steel has no fire resistance, and a steel building *can* be destroyed by a fire.

2.7.1.4.1 Steel Bar Joists

Steel bar joists were noted to fail under ASTM E119 standard test conditions at approximately 7 minutes. It was noted that this should be compared with wood joists, which lasted 10 minutes. Another concern regarded bar joist construction with wide on-center spacing that is typically found in a roof system. Firefighters who cut a hole for ventilation may find themselves standing on only a thin piece of corrugated steel, which could bend or twist, causing a fall. Bar joists are also noted to be an extremely strong structural member, using very small steel sections. The long spans and high strengths require the use of ties to resist overturning. However, in a fire, these ties may cause multiple truss failures.

2.7.1.5 Other Building Components Not Considered to Be Lightweight

There is a separate model code classification for heavy timber construction, including solid wood or glued-laminated members, typically 6 inches or greater in width and 10 inches or greater in depth. Actual sizes allowed are prescribed by the code, and may be slightly less in some cases and greater in others. Heavy timber is also often referred to as "mill" construction. During a fire, heavy timber construction resists failure longer than a conventional wood frame structure because the structural members are larger, have a smaller surface-to-mass ratio, and take longer to burn. As a wood member burns, a layer of char develops which acts like insulation, slowing the rate of burning. These wood members continue to carry structural loads by virtue of the mass of the unburned wood. This concept applies to all wood members, with heavy timber being the most durable, due to its having a greater mass of wood.

2.7.1.5.1 Heavy Timber Construction

This type of construction is very durable in the early stages of a fire. The only problem with this construction type is that radiating heat from total involvement of a large building may prevent firefighters from getting close enough to the building to fight the fire, and allow the fire to spread to adjacent buildings. After hours of burning, the building will eventually collapse.[14]

2.7.1.5.2 Glue-Laminated Beams

It is stated that glue-laminated beams will collapse early, due to the adhesive bond deteriorating under fire. The adhesives can be expected to burn at a relatively low temperature, accelerating system failure. With laminated beams, one must also expect early collapse.

2.7.1.6 Concealed Spaces

The concern with concealed spaces is the rapid spread of fire throughout the truss void. There is also concern that a concealed space will be used for storage and other uses, which may increase the load on the trusses, causing earlier collapse.

2.7.1.7 Testing of Fire Assemblies

Testing done on fire-rated assemblies is of concern because it does not represent the actual performance of components under realistic fire conditions. The reasons behind earlier-than-expected collapse of tested assemblies are:

- The actual fire may be more intense than a test fire.
- The test specimen may not allow for prediction of the performance of an actual floor.
- The fire may have burned undetected for longer than the test period.
- The actual building may not be constructed as well as the test specimen.
- Testing does not take into account penetrations of the gypsum.
- Testing does not provide for additional air being available through poor fire- and draftstopping.
- Impact loading due to a firefighter's weight is not taken into account.

Finally, it is noted that tests are also sponsored by organizations with a vested interest in the result; therefore, the construction is perfect. This level of construction quality is probably not performed in the field. There may be other time/temperature curves that more accurately reflect real fire conditions. Their use should be considered in the future.

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[14] After hours of burning most, if not all, buildings will collapse.

2.7.1.8 Building Codes

Firefighter safety has never been addressed in building code regulations. The responsibility is left to the fire department. Building Codes are not self-enforcing; therefore, fire-safe measures can be violated, or disregarded. Many buildings within the model building code regulations are allowed to be built unprotected from the attack of fire. In these cases, both combustible and non-combustible buildings are expected to collapse in a fire, and may be subject to early failure as well. Finally, it is noted that no building code provision provides for the personal safety of the firefighter.

2.7.1.9 Warning Signals

With new lightweight components, firefighters cannot rely on outdated warning concepts such as:

- The floors will sag.
- The floors or roofs will soften.
- Water will flow through bricks.
- Smoke will puff out of mortar joints.
- Strange noises will take place.

It was noted, however, that often floors or roofs will begin to feel spongy—an indication of a problem. However, warning signals may not always be observed as predicted, and recognized soon enough to warn everyone. This is cause for concern.

2.7.1.10 Tactical Considerations

It is unwise to assume what is not known when making tactical firefighting decisions. It is equally unwise to assume all construction behaves like wood joists. Therefore, there is no substitute for pre-fire plans and follow-up inspections. This information can be used in suppression tactics and factored into fire ground operation procedures based on the collapse potential of the building. Then tactics can reflect the conditions, construction and materials encountered.

New ventilation procedures must be given consideration. Using safe working practices on the roof or venting walls and windows may be more appropriate. A safe rule is that a fully involved building is about to collapse. If the fire begins to burn the structural components, firefighters do not belong on the roof, or on or beneath the floors. Key tactical information required includes knowing when the fire begins to burn structural components and how long the fire has been burning.

2.7.1.11 General Fire Safety

The degree of risk that will be accepted for building occupants is a difficult decision, at best. Given this, fire safety can be broken into three primary areas of concern: life safety, property protection, and continuity of operations—in that order of importance.

2.7.2 Industry Literature

A SUMMARY OF THE ISSUES EXPRESSED IN LITERATURE FOUND OUTSIDE OF FIRE SAFETY COMMUNITY ON THESE TOPICS FOLLOWS:

2.7.2.1 Truss Plate Connectors

It has been suggested that these connectors reflect heat during the fire, and actually protect the wood below the connector themselves. There are photographs and a videotape of a connector plate under fire conditions that show a period of time where the truss plate *does* protect the wood below due to the reflection of radiant energy. The plate eventually does conduct heat in the wood below it, causing pyrolysis to take place. This reduces the strength of the connection until it fails.

There is also a concern that truss plates loosen during a fire. Photographs indicate this may occur to some degree, but the reason behind this occurrence is unknown.

2.7.2.2 Other Connections

Testing has shown that finger joints in lumber retain 75% of their strength in a fire, as compared to identical pieces of solid lumber.

2.7.2.3 Testing for Fire Performance

It was noted that ASTM E119 should be used only as a measure of comparative performance of various test assemblies under standardized test conditions. To use these test results to predict the performance of actual building construction requires careful evaluation of the test conditions.

A 1-hour rated assembly is expected to perform for only the one hour time period—nothing more, nothing less. However, this rating does not mean the assembly will last for one hour during a "real" fire exposure.

Finally, the ASTM E119 test method has been judged by knowledgeable experts to represent a relatively severe fire for use in assessing the adequacy of the tested assembly in providing life safety protection. It would be impossible to test every specific assembly type under all possible fire conditions. A representative sampling is the best approach.

2.8 Summary

THE FOREGOING PROVIDES A COMPENDIUM OF THE POINTS OF VIEW AS EXPRESSED IN THE LITERATURE. ANALYSIS OF THESE COMMENTS IS FOUND IN **CHAPTER 7: DISCUSSION**.

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Chapter 3: Fire Loss Statistics

To assess the concerns expressed in firefighting literature, it is important to review known information. This includes available data and statistics on the fire performance of lightweight component construction, as well as results of physical testing of the construction type in question. A review of the relevant statistical information is included in this chapter. A review of physical testing data appears in **Chapter 4**.

In order to have a base from which to perform a risk assessment in the future, and to provide a guide with which to focus efforts on areas that are critical from a fire endurance perspective, it is helpful to review the statistics surrounding this issue. This information can provide a view of the magnitude of various aspects of fire loss, as well as clarify issues that require further review.

3.1 One- and Two-Family Dwelling Fires

A view of the fire problem in the United States can be obtained by defining where that fire problem exists. Seventy-five percent of the fire-related fatalities in 1988 occurred in residential properties. Five percent were in non-residential properties. Sixty-seven percent of fire-related injuries in 1988 occurred in residential properties with 13% in non-residential properties. These data are shown in the two Figures below, and are virtually the same as data for 1983.

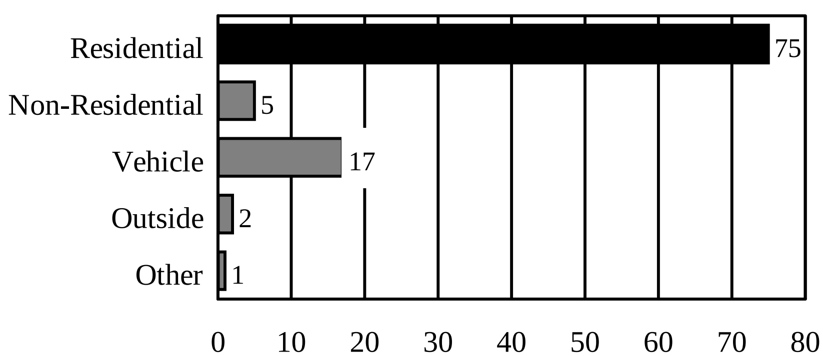


Figure 2.   Percent Fatalities[1]

[1] Federal Emergency Management Agency (FEMA), **Fire in the United States**, 7th ed., August 1990.

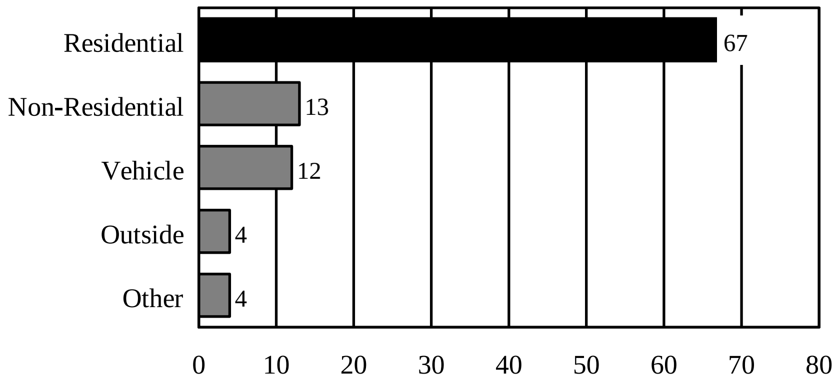
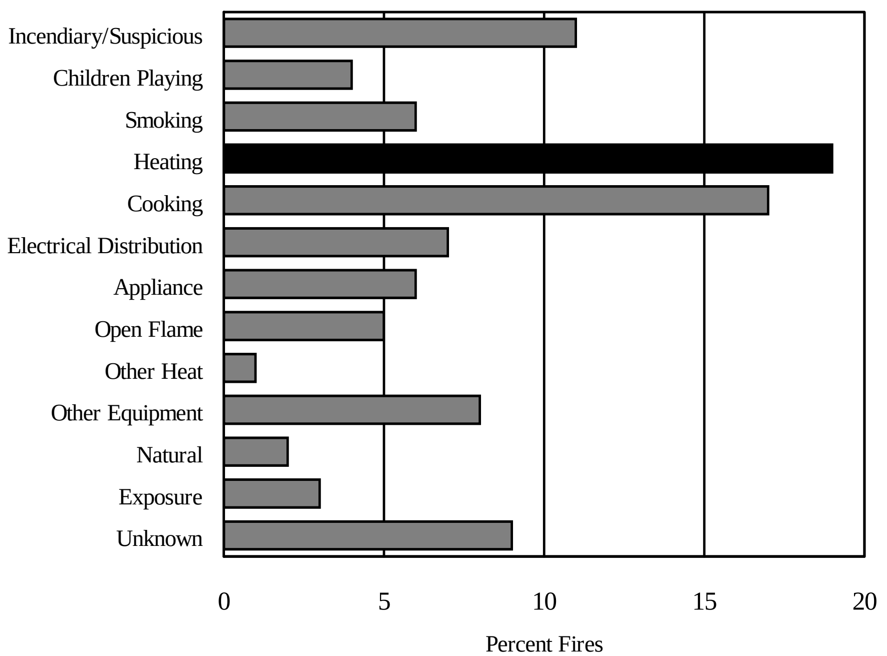


Figure 3. Percent Injuries[2]

Figure 4 below details the leading causes of residential fires in 1988.[3] A similar trend is seen in the 1983 data.[4]



Raw%	Adjusted%
11	12
4	5
6	7
19	22
17	19
7	8
6	6
5	6
1	1
8	9
2	2
3	3
9	

Figure 4. Cause of Residential fires[5]

Heating fires are those where the equipment involved in ignition includes: central heaters, fireplaces, portable space heaters, fixed-room heaters, wood stoves, and water heating. The

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[2] Ibid.

[3] FEMA, **Fire in the United States**, 6th ed., July 1987.

[4] FEMA, **Fire in the United States**, 7th ed., August 1990.

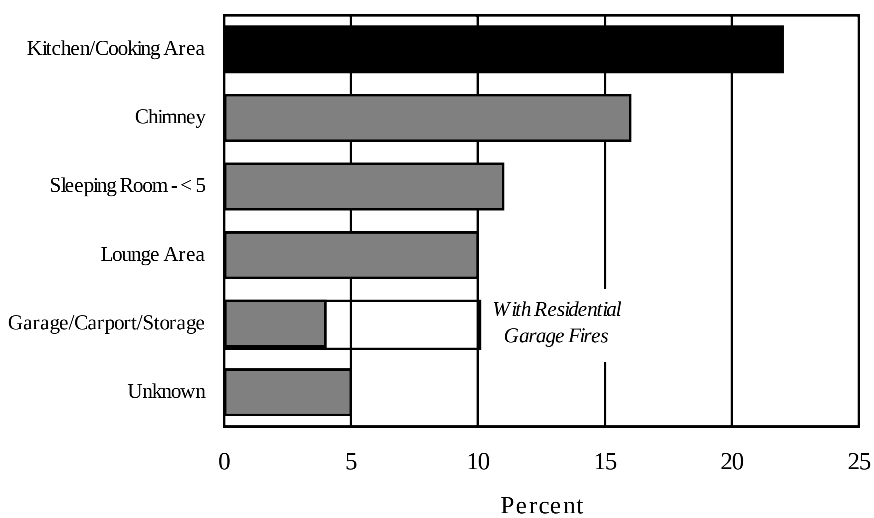
[5] Source: National Fire Incident Reporting System (NFIRS)

central- and water-heating portions of the problem have remained relatively unchanged over the years, while fires due to portable space heaters, wood burning stoves and chimneys rose very sharply from the late 1970s to the early 1980s, then subsided somewhat.[6]

Cooking—the second leading cause of residential fires—was the leading cause of fires in the 1980s, but was passed by heating with the surge in use of alternative space heaters and wood heating in the late 1970s. Cooking is by far the leading cause of fire injuries. Most cooking fires come from unattended cooking, rather than equipment failures.[7]

It is assumed most often that arson (incendiary/suspicious fires) is a crime against businesses; in fact, the statistics indicate that there is a very large arson problem in the home. The causes range from vandalism fires set by youths and revenge fires set to end quarrels, to fraud against landlords or insurance companies. Residential arson fires are set most often in bedrooms.

Additional insight into residential fires is gained by looking at the leading rooms of origin for fires in one- and two-family dwellings (see Figure 5). This is virtually the same as data from 1983.



Raw%	Adjusted %
22	28
16	17
11	12
10	11
4	4
5	

Note: The white bar for garage fires indicates approximately how large they would be if the residential garage portion of storage fires was added here. All of the other bars would decrease and would have to be re-computed because the added garage fires would increase the total number of fires by 6 percent.

Figure 5. Leading Rooms of Fire Origin for Residential Structures [8]

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[6] Ibid.

[7] Ibid.

[8] Ibid.

Fires - 544,000 **Civilian Fatalities - 3,900** **Civilian Injuries - 14,100**

	Percentages		
Area of Origin (901 Code)	**Civilian Fatalities (For Ranking)**	**Fires**	**Civilian Injuries**
Living room, den, lounge (4)	40.2	11.6	21.9
Bedroom (21-22)	24.1	11.6	20.9
Kitchen (24)	14.0	20.6	27.5
Structural Area (70-79)	**5.8**	**15.5**	**7.4**
[Crawl space (71)]	(1.5)	(3.2)	(2.9)
[Unspecified (79)]	(1.0)	(1.0)	(0.7)
[Balcony, porch (72)]	(0.9)	(1.1)	(0.9)
[Ceiling/Floor Assembly (73)]	(0.7)	(0.8)	(0.5)
[Ceiling/Roof Assembly (74)]	(0.6)	(2.3)	(0.7)
[Wall Assembly (75)]	(0.6)	(2.0)	(0.8)
Dining room (23)	2.3	1.1	1.6
Heating equipment room (62)	1.9	3.7	3.6
Bathroom (25)	1.2	1.7	1.9
Hallway, corridor (01)	1.2	0.9	1.1
Garage* (47)	1.1	3.4	3.7
Interior stairway (03)	1.0	0.4	0.4
Closet (42)	0.9	1.2	1.3
Other known single area	**4.2**	**26.6**	**7.5**
[Chimney (51)]	(0.4)	(18.9)	(0.7)
Multiple areas (97)	0.8	0.7	0.6
Unclassified, not applicable (98-99)	1.3	1.0	0.6
Total	**100.0**	**100.0**	**100.0**

* Does not include dwelling garages coded as property type, which is a larger number.

Table 1. Annual Averages of Fatalities and Injuries in One- and Two-Family Dwellings and Mobile Homes, 1980-1984 [9]

Table 1 (above) provides even greater detail, and shows that fires originating in structural areas made up 15.5% of fires during the study period. Of all fires, 0.8% started in a floor/ceiling assembly area and 2.3% started in a roof ceiling assembly area. Fires that began in a concealed floor or roof space or crawl space caused 2.8% of the civilian fatalities and 4.1% of civilian injuries. 81.8% of the civilian fatalities and 73.8% of civilian injuries occur in fires that start in main living areas of residential structures.

[9] NFPA Standard 13D, 1989 Ed.

The leading areas of fire origin, taken from a more recent study, are shown in Table 2. Here, fires began in structural areas less than two percent of the time. Forty-nine percent of the time fires began in a living area that typically would be compartmentalized.[10]

Area of Home	Heating	Cooking	Incendiary	Electrical Distribution	Smoking	Children Playing	Total
Lounge	5,442 13.1%		2,116 13.5%	1,529 12.4%	1,919 25.8%	698 10.6%	11,704 11.0%
Sleeping Under 5	1,160 2.8%	85 0.4%	2,778 17.7%	2,333 18.9%	2,957 39.8%	3,122 47.6%	12,435 11.7%
Kitchen/Cooking	1,037 2.8%	22,416 95.0%	1,218 7.7%	1,400 11.3%	569 7.7%	448 6.8%	27,088 25.4%
Lavatory					282 3.8%		282 0.3%
Closet						355 5.4%	355 0.3%
Garage/Carport/ Vehicle Storage		97 0.4%		631 5.1%	199 2.7%	314 4.8%	1,241 1.2%
Chimney	21,524 52.1%						21,524 20.2%
Heating Equipment Area	3,843 9.3%						3,843 3.6%
Exterior Balcony/Open Porch		169 0.7%					169 0.2%
Ceiling/Roof				980 7.9%			980 0.9%
Exterior Wall			932 5.9%				932 0.9%
Court/Terrace/Porch		85 0.4%					85 0.1%
Multilocation/Use			1,048 6.7%				1,048 1.0%
Unknown							25,254 23.7%
Total Fires	**41,286**	**23,322**	**15,706**	**12,342**	**7,435**	**6,559**	**106,650**

Note: For each cause, the five most common rooms or areas of origin reported are shown. Data here are NFIRS raw counts, NOT national estimates. Percentages shown are column percentages (e.g., percentages of heating or cooking fires, not percentages of lounge fires).

Table 2. Leading Rooms of Origin by Cause for One- and Two-Family Dwelling Fires[11]

Finally, a 1986 national survey by the National Association of Home Builders on residential fire fatalities found that newer homes were much safer than older homes: 43 lives were lost in homes less than five years old. In sharp contrast, approximately 4,100 lives, or 89% of all

[10] FEMA, **Fire in the United States**, 7th ed., August 1990.

[11] Ibid.

residential fire fatalities during the study period, occurred in homes that were 20 years old or older.[12]

3.1.1 Observations on One- and Two-Family Dwelling Fires

The significance of the high number of fire-related fatalities in residential properties indicates that the greatest impact can be achieved by solving problems associated with compartments. The issues here include penetrations of protective membranes and concealed spaces, assuring that compartments comply with code-conforming construction techniques, installing the proper rated assembly, residential sprinkler protection, etc. Figure 4 shows that sprinklers placed in the living space could effectively contain many of these fires and reduce losses to civilian lives, property and, consequently, the potential loss of firefighter lives.

Based on statistics, residential fires are the nation's most serious fire problem. Three-quarters of all fire-related fatalities and two-thirds of all fire-related injuries occur in residential properties. Fire and code officials have focused attention on the need for smoke detectors. Getting people out of a burning structure early is the best way to save lives. Also, residential sprinklers could drastically reduce the dollar loss attached to these fires. The application of sprinklers may go a long way toward reducing civilian fatalities and injuries even further.

The foregoing data suggest that the majority of fires begin in areas where there is compartmentation. Fires began within a structural space 3.1% of the time and caused 2.8% of civilian fatalities and 4.1% of civilian injuries. This suggests that most fires originate within compartmentalized rooms where a protective membrane separates the structural system from the fire. In these instances, the performance of the protective membrane will be vital to the performance of the overall structural system in a residential fire.

The key to compartment effectiveness is having the compartment remain intact prior to and during a fire. Any penetration will cause the fire to spread rapidly to other areas of the structure. With proper compartmentation, one can expect a given period of satisfactory performance for structural elements in the majority of fires that occur in residential properties. In many cases, the performance of a compartment can be approximated through calculation methods.

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[12] **Nation's Building News**, October, 1991

3.2 Apartment Fires

A trend similar to that of single-family residential fires is seen for the leading room of origin in apartments (see Figure 6). The exception is that apartments do not have as many chimney fires.

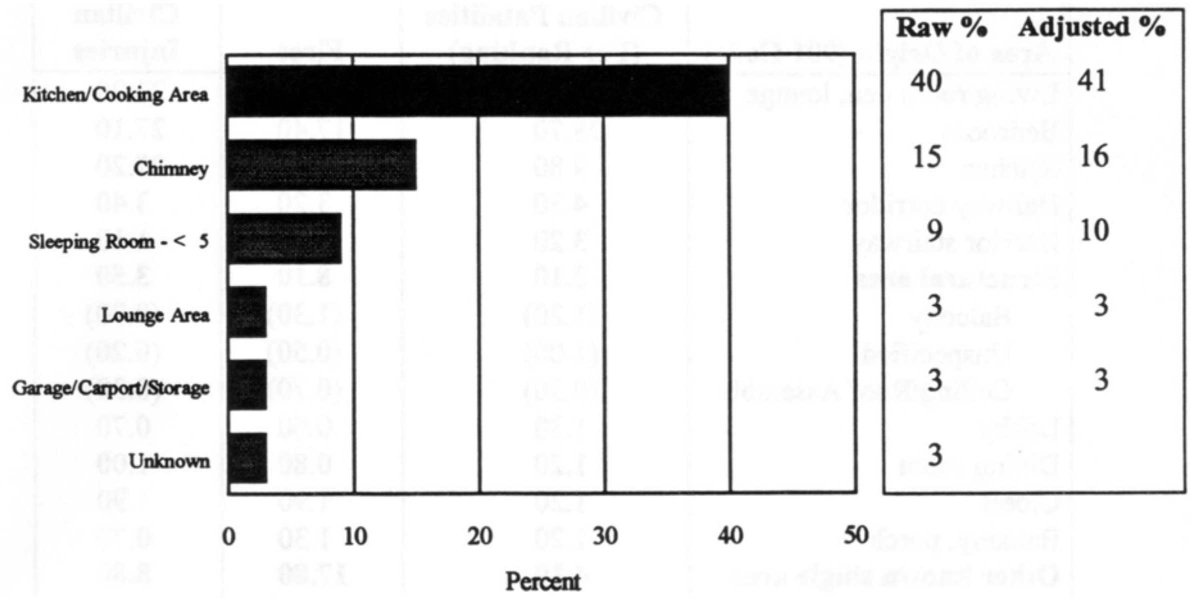


Figure 6. *Leading Rooms of Origin in Apartment Fires, 1987*[13]

[13]FEMA, **Fire in the United States**, 7th ed., August 1990.

In a study shown in Table 3, fires that originated in a structural areas made up 8.1% of all fires.[14] Of these, 0.7% began in a structural assembly area.

Fires - 123,000 **Civilian Fatalities - 930** **Civilian Injuries - 5,470**

	Percentages		
Area of Origin (901 Code)	**Civilian Fatalities (For Ranking)**	**Fires**	**Civilian Injuries**
Living room den, lounge	38.50	11.30	23.20
Bedroom	28.70	17.40	27.10
Kitchen	9.80	35.30	27.20
Hallway corridor	4.30	3.20	3.40
Interior stairway	3.20	1.00	1.10
Structural area	**3.10**	**8.10**	**3.50**
Balcony	(1.20)	(1.30)	(0.70)
Unspecified	(1.00)	(0.50)	(0.20)
Ceiling/Roof Assembly	(0.30)	(0.70)	(0.30)
Lobby	1.30	0.60	0.70
Dining room	1.20	0.80	1.00
Closet	1.20	1.90	1.90
Balcony, porch	1.20	1.30	0.70
Other known single area	**4.10**	**17.80**	**8.80**
Bathroom	(0.60)	(2.10)	(1.30)
Multiple Areas	1.60	0.70	0.90
Unclassified, not applicable	1.80	0.60	0.50
Total	**100.00**	**100.00**	**100.00**

Table 3. Annual Averages of Fatalities and Injuries in Apartments, 1980-1984 [15]

[14] NFPA 13 R, **Installation of Sprinkler Systems in Residential Occupancies up to Four Stories in Height**, 1989 Edition.

[15] Ibid.

A more recent study details the leading rooms of origin in apartment fires (see Table 4).[16]

	Leading Causes						
Area of Home	**Cooking**	**Arson**	**Smoking**	**Heating**	**Children Playing**	**Open Flame**	**Total**
Interior Stairway		308 4.5%					308 0.9%
Hallway		755 10.9%	140 2.6%				895 2.7%
Lounge Area		739 10.7%	1,427 26.7%	379 14.6%	293 11.5%	295 13.2%	3,133 9.4%
Sleeping Under 5	66 0.5%	1,137 16.5%	2,049 38.3%	251 9.7%	1,331 52.2%	460 20.5%	5,294 15.8%
Dining	32 0.2%						32 0.1%
Kitchen/Cooking	13,333 96.4%	444 6.4%	355 6.6%	221 8.5%	193 7.6%	269 12.0%	14,815 44.3%
Lavatory					59 2.3%	195 8.7%	254 0.8%
Closet					194 7.6%		194 0.6%
Trash Area/Container			322 6.0%				322 1.0%
Chimney				281 10.8%			281 0.8%
Heating Equipment Area				660 25.4%			660 2.0%
Exterior Balcony	121 0.9%					88 3.9%	209 0.6%
Court/Terrace/Patio	38 0.3%						38 0.1%
Unknown	241 1.7%	3,520 51.0%	1,061 19.8%	808 31.1%	481 18.9%	932 41.6%	7,043 21.0%
Total	**13,831**	**6,903**	**5,354**	**2,600**	**2,551**	**2,239**	**33,478**

Note: For each cause, the five most common rooms or areas of origin reported are shown. Data here are NFIRS raw counts, NOT national estimates. Percentages shown are column percentages (e.g., percentages of heating or cooking fires, not percentages of lounge fires).

Table 4. Leading Rooms of Origin by Cause for Apartment Fires, 1987[17]

In this study, no fires were recorded as beginning in structural member areas. The fires began in areas that were compartmentalized 70.7% of the time.

[16] Ibid.

[17] Ibid.

3.2.1 Observations on Apartment Fires

Fires beginning within compartments make up the majority of fires in apartments, as is the case with one- and two-family dwellings. Therefore, the same comments apply to apartments as were made about one- and two-family dwelling fires above.

3.3 Non-Residential Fires

In general, the non-residential share of the fire problem is getting smaller, while the residential share is growing. There has also been a dramatic improvement in life safety over the last few years in non-residential structures.[18]

Stores, offices, manufacturing facilities, and storage facilities have the greatest number of fires and dollar loss.[19] The leading causes of non-residential structure fires are shown in Figure 7 below:

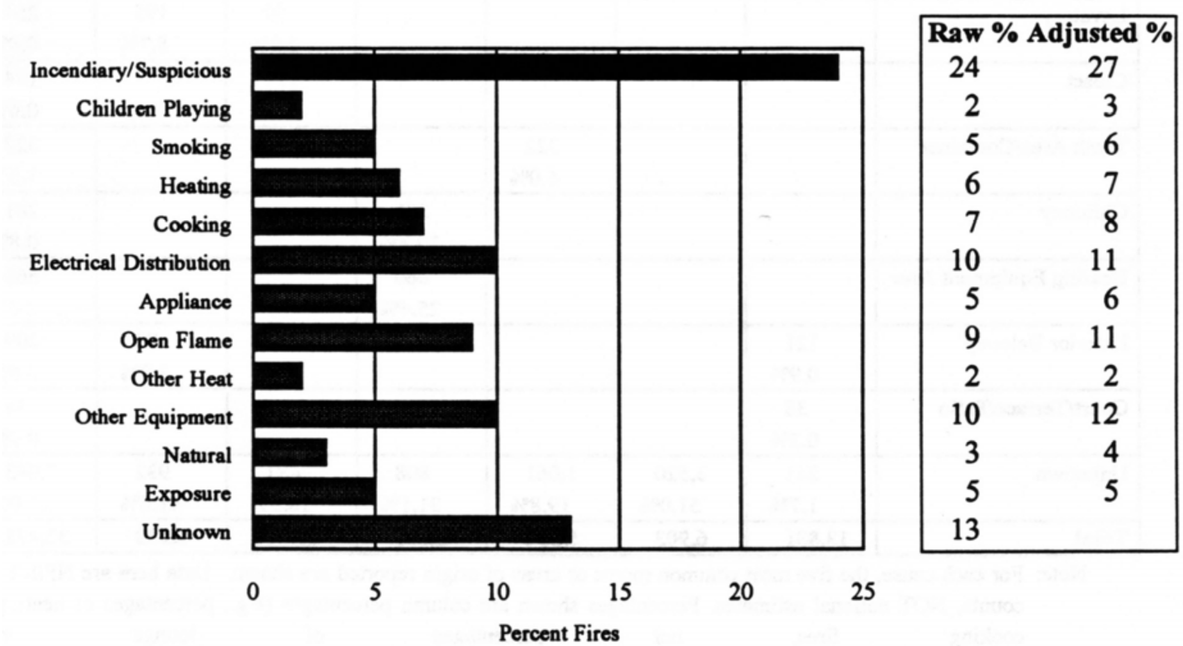


Figure 7. Causes of Non-Residential Structure Fires, 1987 [20]

[18] FEMA, **Fire in the United States**, 7th ed., August 1990.

[19] Ibid.

[20] Ibid.

3.3.1 Observations on Non-Residential Fires

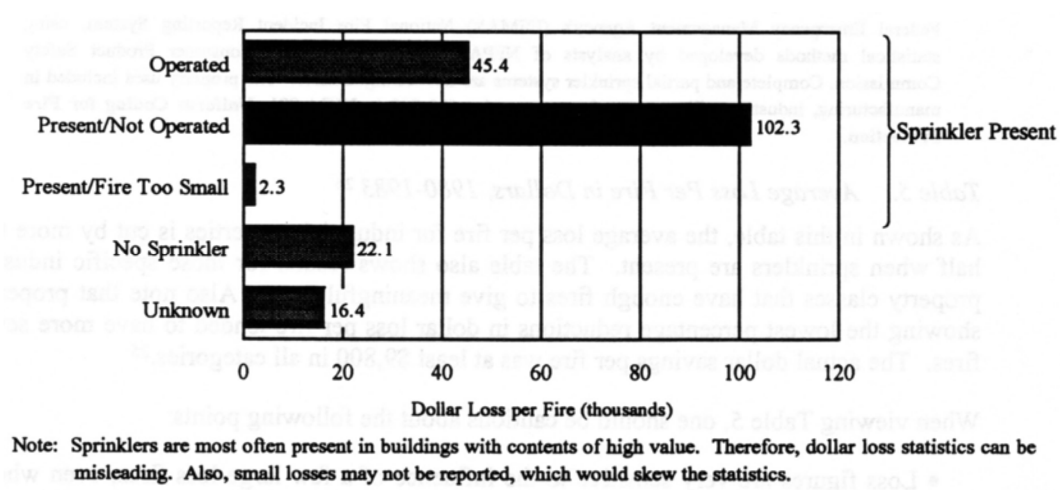
By far, the leading cause of non-residential fires is incendiary[21], which has been the case since the National Fire Incident Reporting System (NFIRS) was started.[22] It is difficult to define which fires are incendiary, since they are set in areas that have easy access, and with the intent to damage or destroy the structure. If these fires are disregarded, the majority of fires are accidental and start in locations that may either be protected by sprinklers or compartmentalized, as can be deduced from the causes listed in Figure 7.

Since the majority of fires in non-residential structures are incendiary, they are probably set outside of normal business hours. This is a benefit, considering there is less likelihood of civilian fatality and injury; but it is worse for firefighters, since these fires would often be set during time periods, and in ways that are less desirable from a firefighting standpoint. For example, the arsonist often uses accelerants and other highly flammable materials to speed the burning process.

When considering life-safety of occupants, given that only 5 to 10% of all fires and 5 to 7% of fire-related fatalities occur in non-residential occupancies, the focus should be on residential construction.

3.4 Sprinkler Performance

Figure 8 relates the performance of sprinklers in terms of dollar loss in non-residential fires.



Note: Sprinklers are most often present in buildings with contents of high value. Therefore, dollar loss statistics can be misleading. Also, small losses may not be reported, which would skew the statistics.

Figure 8. Sprinkler Performance in Non-Residential Structures: Dollar Loss per fire, 1987 [23]

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[21] Often referred to as "arson".

[22] FEMA, **Fire in the United States**, 7th ed., August 1990.

[23] Ibid.

It is clear from Figure 8 that when sprinklers operate properly, damage (as reflected by dollar loss) is reduced by more than 50%. One must be concerned, however, by the magnitude of loss when sprinklers are present and operate. The mitigating factor behind this high amount of loss is that sprinklers are most often present in properties of high value, and with contents of higher value. Historically, sprinklers are not provided in structures which are small in area or relatively low in value (e.g., single-family dwellings).

Statistics provide evidence that automatic sprinklers reduce fire loss in industrial properties. This evidence is shown in Table 5, which shows statistics on the impact of sprinkler systems from 1980 to 1983.

Property Class	**No Sprinklers**	**Sprinklers Present**	**Percent Reduction**
All manufacturing, industry, utility, defense	20,700	8,800	57
Plastic product manufacturing	59,900	36,400	39
Sawmills, planing mills, wood product mills	22,600	12,600	44
Metal product manufacturing	15,100	5,300	65
Motor vehicle manufacturing, assembly	19,000	5,600	70
Paper, pulp, paperboard manufacturing	16,800	4,800	71
Machinery manufacturing	17,700	3,300	81
Furniture, fixture, bedding manufacturing	34,600	4,900	86
Total	**206,400**	**81,700**	**60**

Loss figures are expressed to the nearest hundred. Estimates are based on the annual NFPA survey and the Federal Emergency Management Agency's (FEMA's) National Fire Incident Reporting System, using statistical methods developed by analysts of NFPA, FEMA, and the US Consumer Product Safety Commission. Complete and partial sprinkler systems are not distinguishable. The property uses included in manufacturing, industry, utility, and defense are codes 600-799 in NFPA 901, **Uniform Coding for Fire Protection**.

Table 5. Average Loss Per Fire in Dollars, 1980-1983 [24]

As shown in this table, the average loss per fire for industrial properties is cut by more than half when sprinklers are present. The table also shows results for those specific industrial property classes that have enough fires to give meaningful data. Also note that properties showing the lowest percentage reductions in dollar loss per fire tended to have more severe fires. The actual dollar savings per fire was at least $9,800 in all categories.[25]

When viewing Table 5, one should be cautious about the following points:

- Loss figures are very sensitive to the influence of a few large-loss fires, even when a multiple-year average is used.
- The databases supporting these calculations cannot distinguish complete from partial systems, which may cause an underestimation of the impact of sprinkler systems.

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[24] NFiPA Fire Analysis Division, "Automatic Sprinkler Systems Do Have an Impact in Industry," **Fire Journal**, January, 1987.

[25] Ibid.

- Evidence shows that sprinklered properties tend to be larger than comparable non-sprinklered occupancies, so the implied savings may be even greater than these figures indicate.[26]
- Sprinklered properties may also be better built and maintained from a fire safety standpoint. This may mean that the statistics shown are crediting sprinklers with loss reductions that were actually caused by many factors. This effect tends to overstate the specific impact of sprinklers.[27]

The statistics in Table 5 include only fires reported to fire departments and, as such, may omit some of the most dramatic sprinkler successes. This has also been a problem with sprinkler statistics in the past. Success stories in small- and even medium-size fires were not reported. Where sprinklers were not successful, human error was often the problem: water was shut off, primarily by closed valves; maintenance was inadequate; or water distribution was obstructed in other cases. These reasons were the cause of unsuccessful sprinkler performance in 47% of the cases from 1925 to 1969.[28]

Operation Life Safety, a program of the National Association of Fire Chiefs, monitors sprinkler activations. Information pertaining to sprinkler performance in the United States for the period of 1983 to 1991 is found in Figures 9 and 10, and Tables 6 and 7.

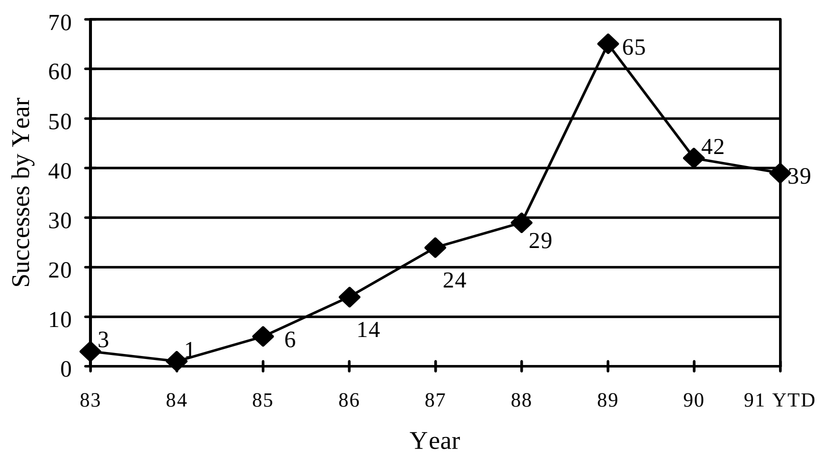

Figure 9. Reported Activations by Year [29]

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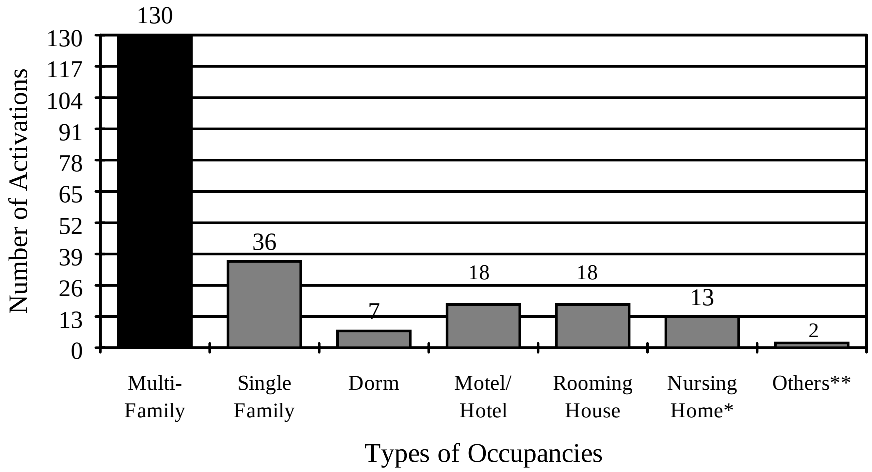
[26] F.E. Rogers, "Fire Losses and the Effect of Sprinkler Protection of Buildings in a Variety of Industries and Trades," Building Research Establishment current paper 9/77, Borehamwood, United Kingdom, February, 1977.

[27] NFIPA Fire Analysis Division, "Automatic Sprinkler Systems Do Have an Impact in Industry," **Fire Journal**, January, 1987.

[28] Ibid.

[29] **Operation Life Safety Newsletter**, 6(12), December, 1991.



* Includes home care, convalescent and retirement home facilities
** Includes high-rise and child-care facilities

Figure 10. Reported Activations by Type of Occupancy, 1983-1991 [30]

Description	# Activations
One-head activations	165
Two-head activations	15
More than two-had activations	2
Not Reported	41

Table 6. Sprinkler Activations Per Fire, 1983-1991 [31]

Room of Origin	# Activations	Percent
Kitchen	86	38.6
Bedroom	33	14.8
Living room	20	8.9
Closet	10	4.4
Laundry room	8	3.5
Storeroom	6	2.7
Bathroom	6	2.7
Garage	3	1.3
Basement	3	1.3
Dining room	2	0.9
Chimney	1	0.4
Others	17	7.6
Not Reported	28	12.5
Total	**223**	

Table 7. Room of Origin, 1983-1991 [32]

[30] Ibid.

[31] Ibid.

[32] Ibid.

Residential sprinklers are also becoming more prevalent, and have been shown to be an effective way to reduce fatalities in home fires.[33] Cobb County, Georgia, a suburb of Atlanta, alone has recorded more than 18 residential fires that were successfully controlled by sprinklers. It is estimated that these fires could have produced at least 17 fatalities had the sprinklers not been present. Another incident involved a fire that occurred in a group home for the developmentally disabled. One of the residents left a lighted cigarette in a closet. At approximately 2 a.m., a sprinkler head in the closet activated and set off an alarm. The fire burned out the closet door, but was successfully extinguished by the sprinkler. This scenario is a prime example of a potential multi-victim incident averted by sprinklers.

Obviously, firefighter safety is enhanced by the presence of sprinklers. Since most fires are controlled by the activation of one sprinkler head, the fire never gets to a size that is dangerous. This contributes to fire ground safety.

Almost $4 billion in residential property was lost in fires in 1989. A 1982 study of sprinklered and unsprinklered dwellings by the City of Scottsdale, Arizona and the U.S. Fire Administration showed property savings of 85% when automatic sprinklers were present and operated in the residence.

3.4.1 Observations on Sprinkler Performance

It is interesting to note that of all the sprinkler activations shown in the above figures and tables, one head usually controlled the fire. Also, the room of origin for these fires was consistent with those shown in the statistics in the previous section. Generally, the room of origin is in an area that is compartmentalized and a primary living area, such as the kitchen, bedroom or living room. This further suggests that the focus ought to be on protected lightweight building components and the various fire performance aspects of this construction method, including concealed spaces.

There is no question that sprinklers can be important in diminishing the impact of fires in any type of construction. It is proven that sprinklers reduce property loss and life loss. There is also a strong possibility that sprinklers could reduce firefighter fatalities, since they contain, and even extinguish, fires prior to arrival of the fire department. Sprinklers are currently the most pro-active fire safety approach in building construction.

3.5 Civilian Fire Casualty Statistics

As shown in Figures 11-14 below, the trends for civilian fire fatalities and injuries have been consistent during the period of 1983 through 1988. Fatalities per million population in the United States averaged 24.8 per year and injuries per million averaged 120.7 per year for this period. The fatalities and injuries are slightly less for 1990 at 20.7 fatalities per million and 114 injuries per million. The trend for civilian fatalities is clearly decreasing when data from 1974 to 1983 are considered.

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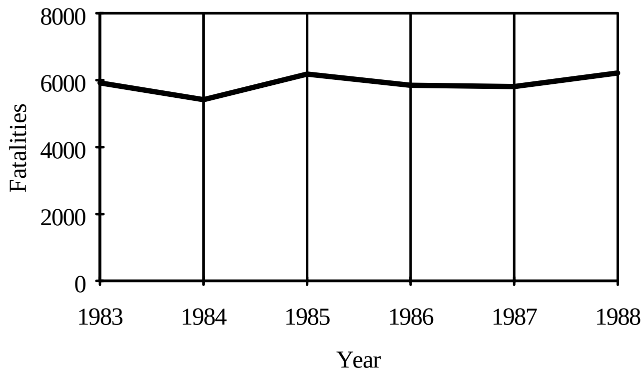
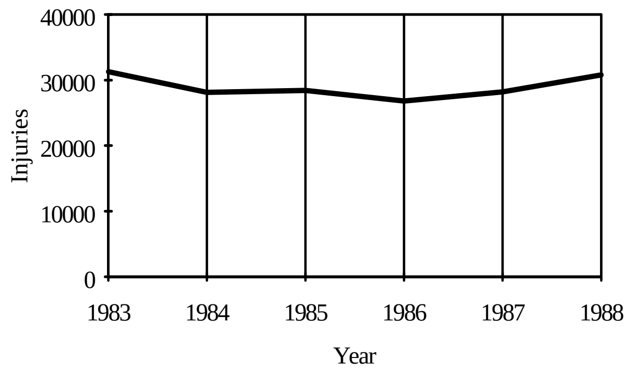
[33] M.J. Dittmar, "Residential Automatic Sprinklers: Grassroots Initiatives," **Fire Engineering**, June, 1991.

To put this in a comparative context, the yearly average for fatalities and injuries per million population in automobile accidents are 188 and 21,307, respectively, for the same period.



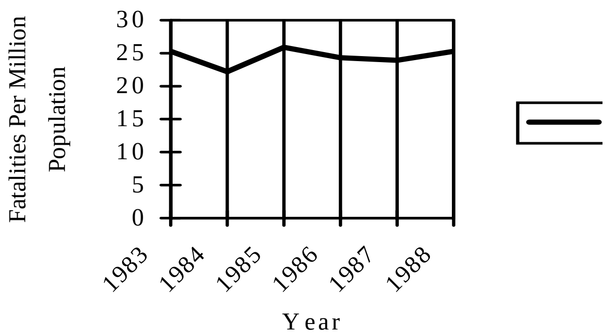
1983	1984	1985	1986	1987	1988	6-Year Change
5,920	5,420	6,185	5,850	5,810	6,215	+ 5.0 %
Average = 5,870						

Figure 11. Civilian Fire Fatality Trend [34]



1983	1984	1985	1986	1987	1988	6-Year Change
31,275	28,125	28,425	26,825	28,215	30,800	- 15 %
Average = 28,944						

Figure 12. Civilian Fire Injury Trend [35]



1983	1984	1985	1986	1987	1988	6-Year Change
25.3	22.2	25.9	24.3	23.9	25.3	0
Average = 24.8						

Figure 13. Civilian Fatalities Per Million Population [36]

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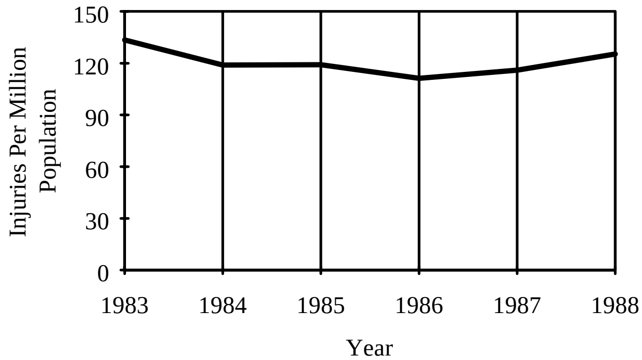
[34] FEMA, **Fire in the United States**, 7th ed., August 1990.

[35] Ibid.

[36] Ibid.



1983	1984	1985	1986	1987	1988	6-Year Change
133.5	118.9	119.1	111.3	115.9	125.3	- 6.1 %
Average = 120.7						

Figure 14. Civilian Injuries Per Million Population [37]

3.5.1 Observations on Civilian Statistics

Fortunately, the trend in fire fatalities and injuries is decreasing. It is surmised that smoke detectors and fire safety education measures are beginning to work. There is also the possibility that construction is safer in a fire due to better electrical distribution systems, construction materials, codes, etc.

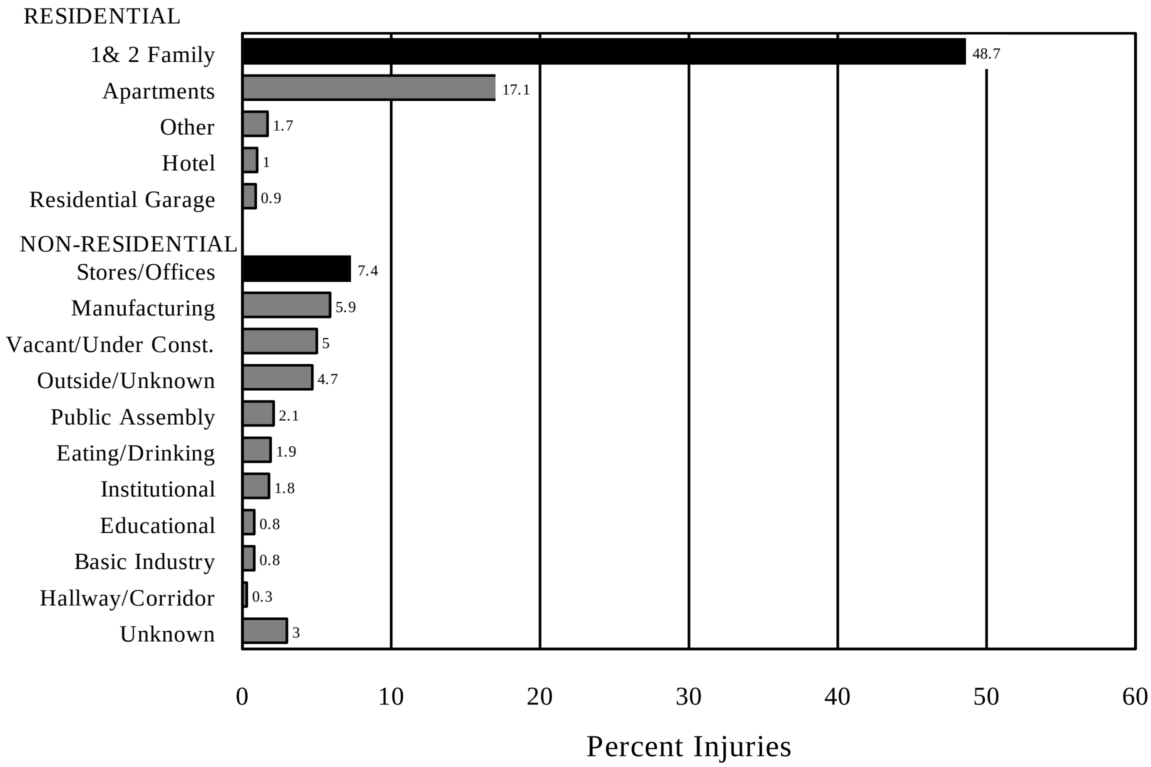
3.6 Firefighter Casualty Statistics

It is very important that the influence lightweight components have on firefighter fatalities and injuries be considered, since this is the primary concern that instigated this study. This will also be important when assessing the cost of solutions in relation to overall risks, and in performing a risk assessment.

3.6.1 Firefighter Injury Statistics

The statistics pertaining to firefighters reveal that fires in residential occupancies account for 67% of all firefighter injuries (See Figure 13 below).

[37] Ibid.



Note: "Public Assembly" here excludes Eating/Drinking, which is shown separately. "Storage" here excludes residential garages, which is shown separately. "Outside/Unknown" here excludes "Vacant/Under Const.," which is shown separately.

Figure 13. Firefighter Injuries Detailed by Property Type, 1987 (Structure Fires Only)[38]

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[38] Ibid.

However, when firefighter injuries per 1,000 fires are detailed, non-residential construction is shown to be more dangerous to firefighters than residential construction (See Figure 13).

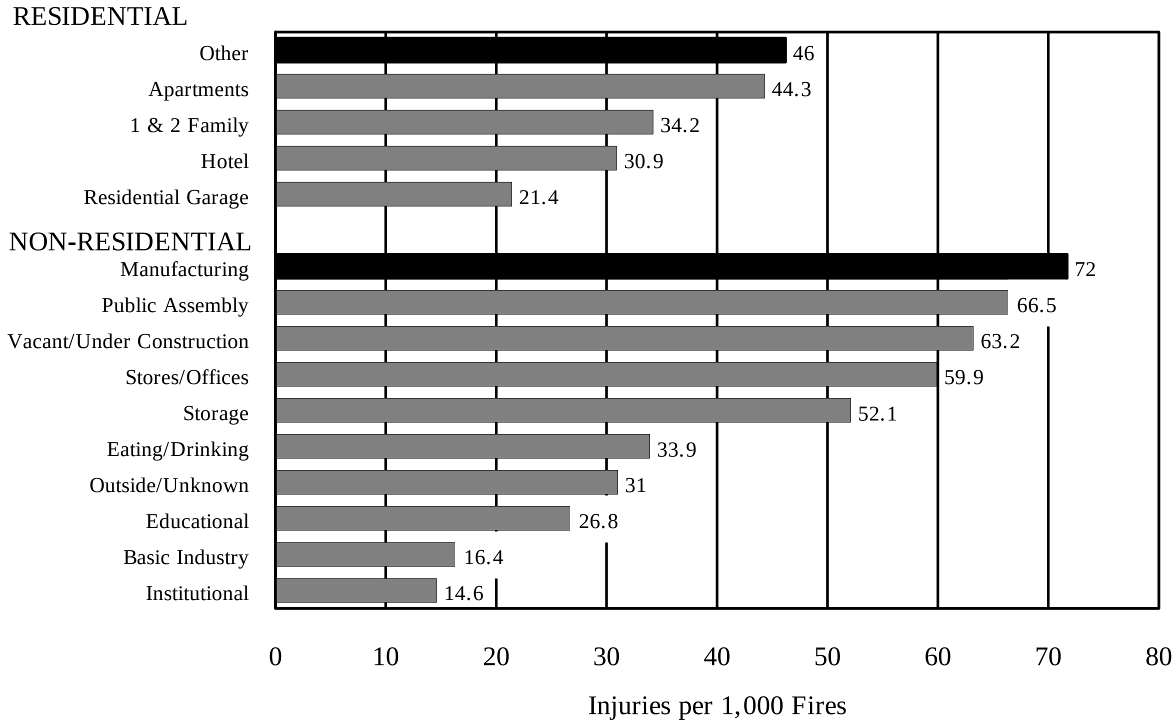

Figure 14. Firefighter Injuries per 1,000 fires by Type of Property, 1987 [39]

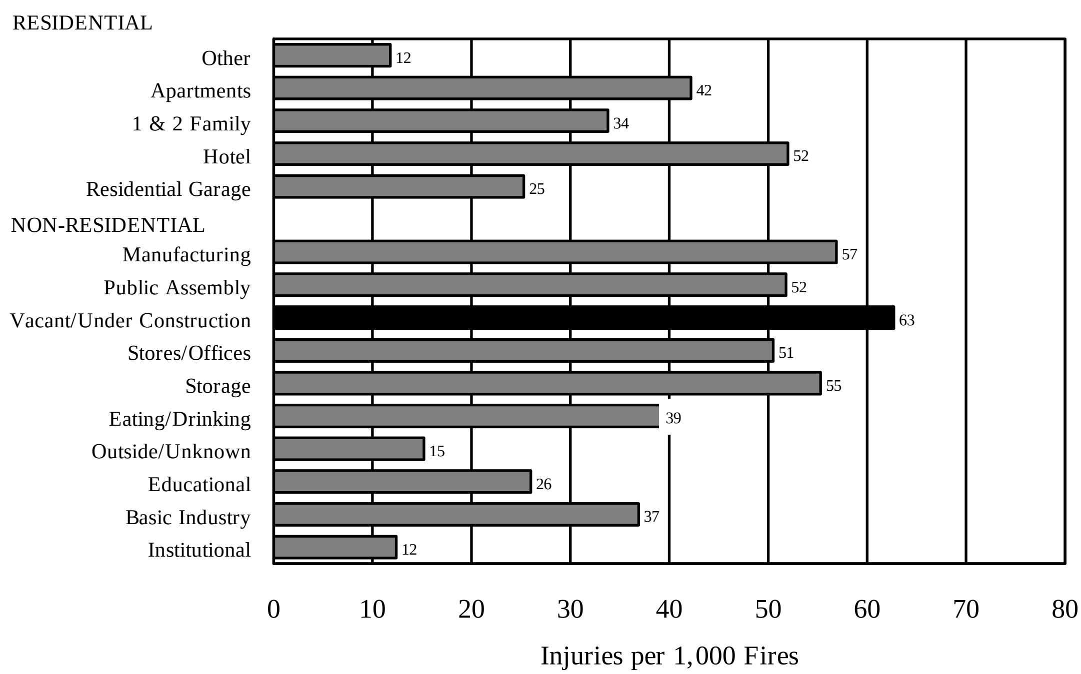

Figure 15. Firefighter Injuries per 1,000 fires by Type of Property, 1988 [40]

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[39] Ibid.

[40] Ibid.

3.6.2 Firefighter Fatality Statistics

Figure 16 shows the number of firefighter fatalities for each year from 1977 through 1990.

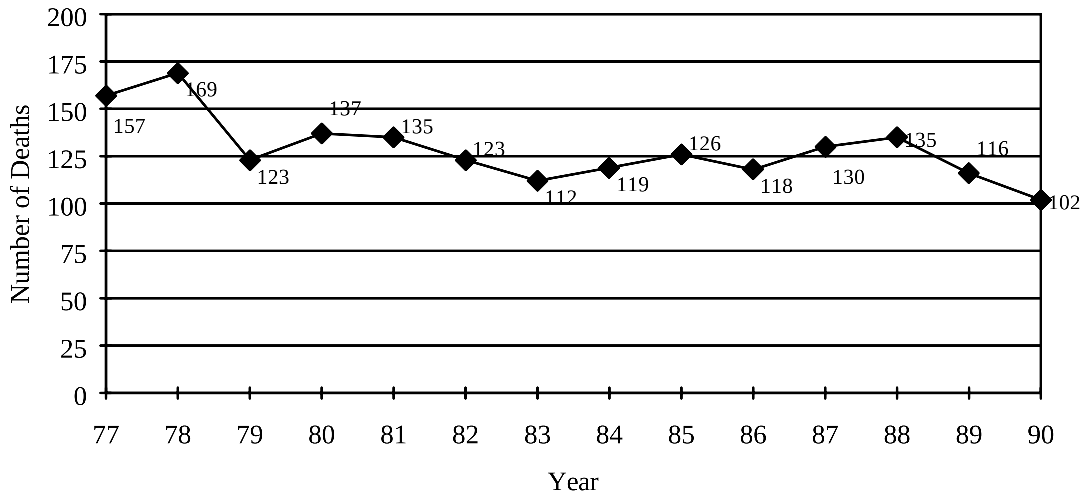

Figure 16. Firefighter Fatalities 1977-1990 [41]

As can be seen, there is a downward trend in firefighter fatalities. Why this is so is not immediately apparent from the literature. One could surmise that firefighters are staying more physically fit, are taking more safety precautions, are better educated on fire ground techniques, etc. This may also be due to the fact that building codes are continuously being upgraded to add new life safety measures, and construction materials and methods are improving, which may result in greater firefighter safety on the fire ground.

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[41] Washburn, AE, LeBlanc, PR, and Fahy, RF, "Report on Fire Fighter Fatalities," **NFPA Journal**, July/August 1991, p. 47.

Figure 17 details firefighter fatalities by type of duty in 1990. Of all on-duty firefighter fatalities, 43.1% were on the fire scene where the structure could have contributed to the loss of life.

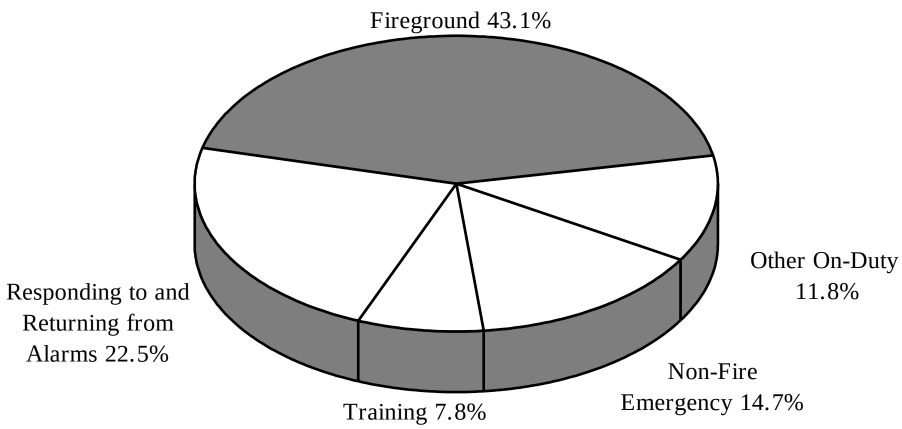

Figure 17. Firefighter Fatalities by Type of Duty, 1990 [42]

To gain a better sense of firefighter fatalities and their causes, data were reviewed from **Fire Command Magazine** Fire Incident Reports from 1980 through 1989. Each of the fatalities detailed were reviewed for cause. The statistical breakdown is detailed in Table 8 and Figures 18 and 19.

	Cause						
Year	**Fatalities**	**Heart Attack**	**Fell or Struck by Object**	**Structural Collapse**	**Exposure to Fire Products**	**Electrocution**	**Other Conditions**
1989	110	59	9	7	6	3	26
1988	129	51	5	17	2	2	52
1987	124	62	6	3	4	0	49
1986	113	58	13	2	8	1	31
1985	119	48	12	7	5	1	46
1984	116	38	15	3	7	2	51
1983	106	52	10	3	6	1	34
1982	117	54	8	12	8	2	33
1981	123	64	7	2	5	0	45
1980	134	60	11	6	7	1	49
TOTAL	**1191**	**546**	**96**	**62**	**58**	**13**	**416**
PERCENT	**100%**	**45.84%**	**8.06%**	**5.21%**	**4.87%**	**1.09%**	**34.93%**

Table 8.   Firefighter Fatalities Taken From Fire Command Magazine, 1980-1989 [43]

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[42] Ibid.

[43] Fire Command statistics compiled by the NFPA Fire Analysis and Research Division. Prepared by authors

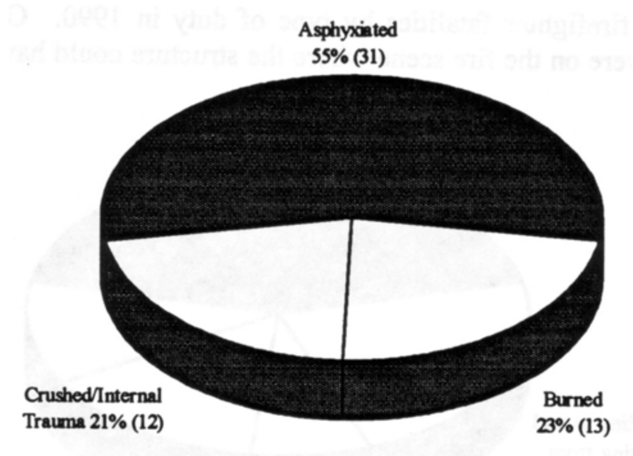

Figure 18. Firefighter Fatalities by Nature of Injury, 1983[44]

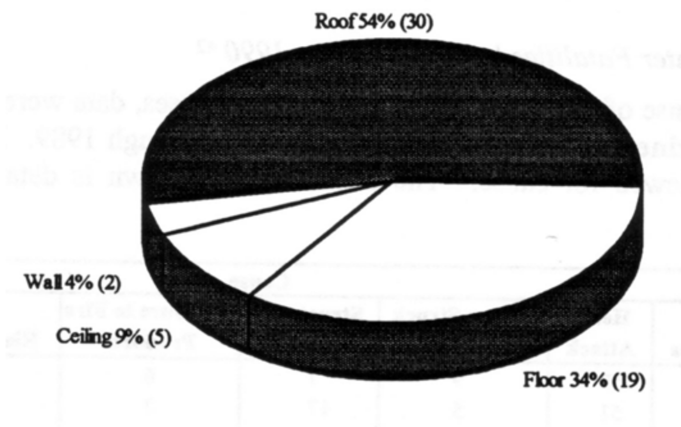

Figure 19. Firefighter Fatalities by Nature of Injury, 1990[45]

The structural collapse cause of fatality data shown in Table 8 was further broken out when the incident report stated specifically that the cause of fatality was due to structural collapse. This includes any conditions that would allow even an inference that the cause of fatality was by structural collapse. For example, a ceiling collapse was included in the structural collapse category, yet it was unknown whether it was the structural supporting member that collapsed, or simply the ceiling material. Therefore, when there was enough detail in "Fell or Struck By Object" (again from Table 8) to place it into the structural collapse category, this was done. This is believed to provide a more realistic picture of structural collapse-related fatalities. This detailed breakdown is shown in Table 9.

---

[44] Source: NFIRS

[45] FEMA, **Fire in the United States**, 7th ed., August 1990.

Year	Total Fatalities	Non-Comb. Wall	Wood Frame Products	Ordinary Roof/Floor[q]	Non-Combust. Roof/Floor	Light Frame Wood Trusses[a]	Timber Trusses	Comb. Wall
1980	134	1.0	3.0	1.0		1.0[f]		
1981	123	1.0		1.0				
1982	117	5.0	1.0	4.0	2.0[o]			
1983	106	1.5[h*]			1.5[hn*]			
1984	116			2.0		1.0[e]		
1985	119	1.0[l*]	2.0	4.0[l*]				
1986	113		0.5[k*]			1.0[d]		0.5[k*]
1987	124	1.5[g**]	1.5[gj*]					
1988	129	3.5[c*]	6.0[i]		2.0[m]	0.5[c*]	5.0[p]	
1989	110	2.0	2.0		1.0	2.0[b]		
TOTAL	**1191**	**16.5**	**16.0**	**12.0**	**6.5**	**5.5**	**5.0**	**0.5**
PERCENT	**100.0%**	**1.39%**	**1.34%**	**1.01%**	**0.55%**	**0.46%**	**0.42%**	**0.04%**

* In five cases (c,g,h,k,l) more than one failure mode is referenced in the event description.

a Unless otherwise noted, all fatalities are in light commercial structures. Truss type is not defined in the description.

b Assumed metal plate connected trusses in Orange County Gift Shop (Mercantile Occupancy). Description does not say.

c Trusses collapsed causing concrete block wall to fall on a firefighter (Mercantile Occupancy).

d A Johnsonville, South Carolina Church (Assembly Occupancy) Truss roof collapsed. Truss type unspecified.

e An apartment building (Group R-2 occupancy) under construction caught due to a fire placed in an unfinished chimney. Roof truss collapsed. Truss type unspecified.

f A delicatessen/restaurant (Mercantile Occupancy) fire roof truss collapse. Truss type unspecified.

g Wood frame roof collapsed causing concrete block chimney to fall.

h 15,000 $ft.^2$ manufacturing plant assumed to use steel bar joists. Caused brick wall to collapse.

i Assumed wood frame in a single-family residence ceiling collapse.

j 100-year-old wood frame church

k Wood frame structure collapsed causing facade to collapse.

l Wall collapse due to roof collapse. Roof type not designated.

m Collapse of concrete floor on steel beams, 1 Fatality. Steel Beam the other.

n Steel bar joist collapse.

o 4 in. concrete floor poured over original joist floor.

p Hackensack, New Jersey Fire. Bolted Timber Bowstring Girder Trusses.

q Description only says the building was of ordinary (type 3) construction.

Table 9. Cause of Fatality by Collapse/Structural Failure[46]

Table 9 was generated by reading each summary in **Fire Command Magazine**, from 1980 through 1989, and ascertaining the specific structural collapse cause of fatality. Unfortunately, the detail of the incident report is often not specific enough to identify the specific structural product. These were categorized in the wood frame products or ordinary category due to the use of 'wood frame' or 'ordinary' in the incident description.

The total fatalities that appear to be attributable to structural framing of the floor or roof system over the period of 1980 through 1989 are 45, 3.8% of the total firefighter fatalities for this period.

[46] Firefighter fatalities taken from NFiPA **Fire Command Magazine**. Statistics compiled by the NFiPA Fire Analysis and Research Division. Summary prepared by Kirk Grundahl.

A similar study done by the Fire Analysis and Research Division of NFiPA for the Federal Emergency Management Agency (FEMA) in August, 1989, provides specific information on firefighter fatalities in structural collapses. For the purpose of this study, structural collapse was defined as: "The failure of structural members resulting in the collapse of a structure or portion of a structure." Two categories of structural collapse were used: the first when firefighters were caught or trapped by a collapsing roof, wall, floor or ceiling; the second when firefighters were struck by a collapsing roof, wall, ceiling or piece of wall.[47]

The study reported that from 1979 through 1988, 93 firefighters were killed in structure fires as a result of structural collapse. Of these 93 victims, 56 were caught or trapped, and 37 were struck by a collapsing roof, wall, etc. Figure 20 shows the number of firefighter fatalities according to these two categories:

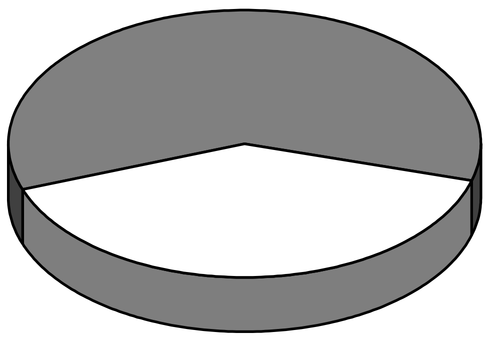

Figure 20. Firefighter Fatality by Category [48]

[47] "Analysis Report on Firefighter Fatalities," Prepared by Fire Analysis and Research Division, NFPA for the Federal Emergency Management Agency, August 1989.

[48] Ibid.

Of the 56 who were caught or trapped by structural collapse, 31 were asphyxiated, 13 died of burns, and 12 died as a result of crushing injuries or internal trauma. These data can be seen in Figure 21.

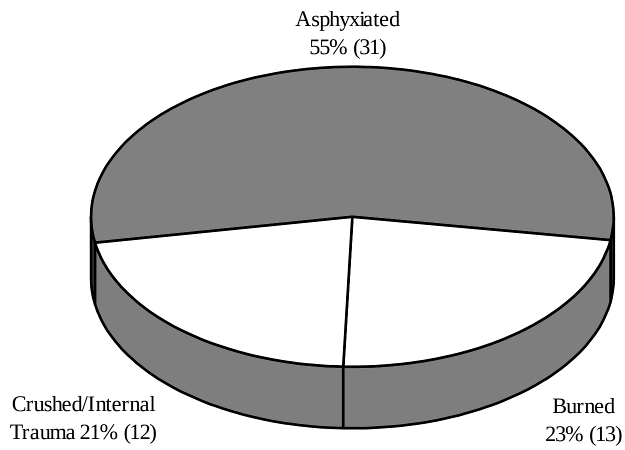

Figure 21. Firefighter Fatalities Resulting From Being Caught or Trapped by a Structural Collapse (56 fatalities) [49]

The building components involved in the collapses were the roof (30 fatalities), floor (19 fatalities), ceiling (5 fatalities), and walls (2 fatalities). These data can be seen in Figure 22.

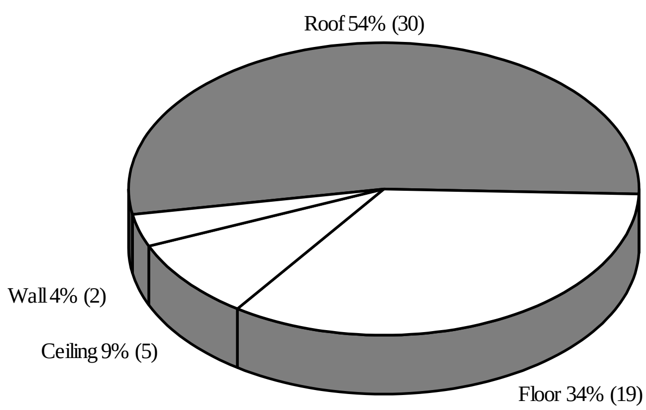

Figure 22. Building Components Involved in Firefighter Fatality [50]

The 30 fatalities in roof collapses occurred as follows: 10 of the victims were on the roof performing ventilation, 17 were inside performing fire suppression activities, 2 were inside pulling ceilings, and 1 was involved in a search for occupants. These data are shown in Figure 23:

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[49] Ibid.

[50] Ibid.

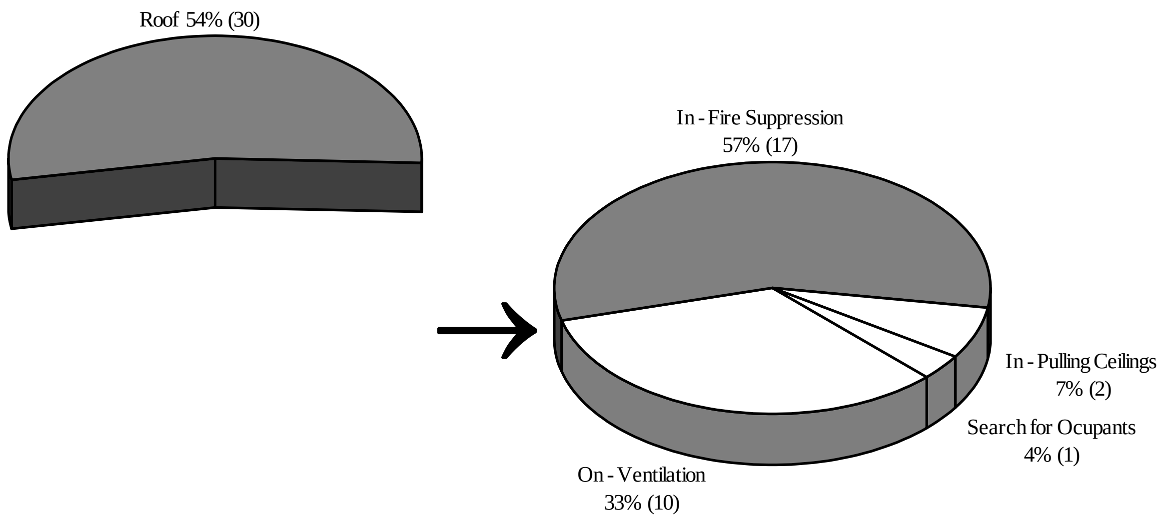

Figure 23. Firefighter Activity During Fatality-Causing Roof Collapse [51]

Figure 24 summarizes the type of occupancy where firefighters were caught or trapped in a structural collapse.

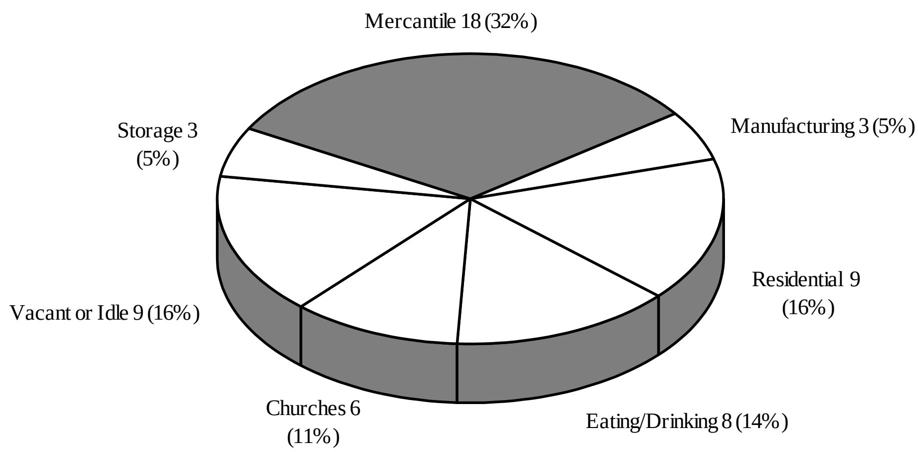

Figure 24. Firefighters Caught or Trapped in Structural Collapses, 1979-1988

Of the 93 fatalities reported in the study, 37 occurred by being struck by a collapsing wall or piece of wall while outside the structure. Of these 37 victims, 30 were operating hand lines (one from an elevated platform) or performing other suppression activities, 3 were killed while escaping from the building, 2 were attempting to move vehicles (in separate incidents), 1 died when a natural gas explosion caused a wall collapse as he and others were attempting to rescue an elderly woman from a fire escape, and 1 was attempting to open a door with a ceiling hook when the wall collapsed on him. These data are shown in Figure 25:

[51] Ibid.

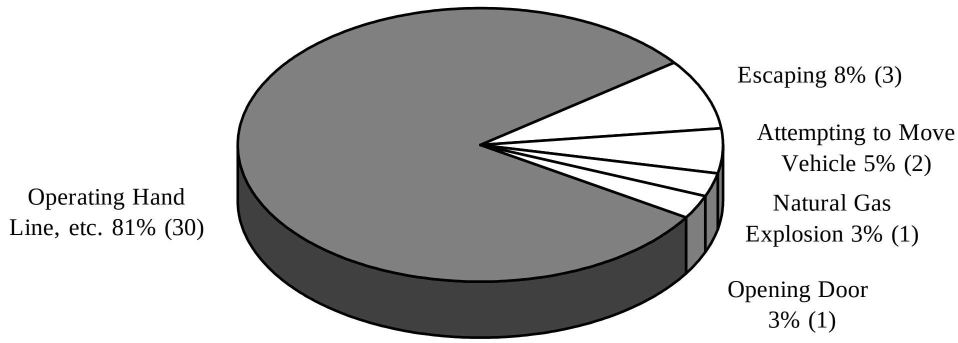

Figure 25. Firefighter Fatalities Caused by Wall Collapse, by Activity (37 fatalities) [52]

In 12 of the wall collapse fatalities described above, the roof was also reported to have collapsed; and in another, the floors collapsed, causing the walls to collapse by being pushed out. The failure of firefighters to maintain an adequate distance between themselves and the building appears to have been a factor in almost all wall collapse fatalities.[53]

Figure 26 summarizes the NFPA study on firefighter fatalities for the period 1979 through 1988:

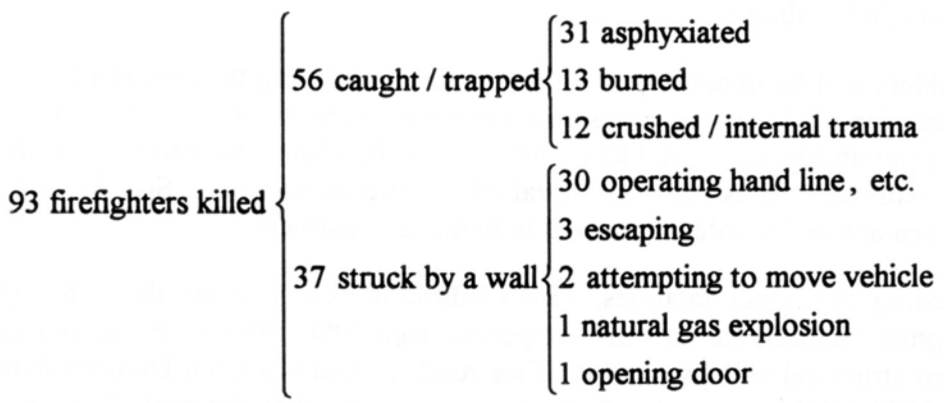

Figure 26. Summary of Firefighter Fatalities 1979 through 1988. [54]

3.6.2.1 Fatalities Due to Truss Roof Collapse

The NFiPA study also identifies collapses involving truss roofs. Seven of these collapses were reported to involve truss roofs. Eleven firefighters died when they were caught or trapped in six of the collapses. The seventh collapse resulted in a firefighter being struck by a collapsing wall after the roof collapsed. The most severe incident occurred in Hackensack, New Jersey, in 1988, when five firefighters were killed when a wood bowstring truss roof

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[52] Ibid.

[53] Ibid.

[54] Ibid.

collapsed.[55] This seems to confirm the numbers developed from Fire Command Magazine as shown in Table 9 above.

3.7 Summary of Fire-Related Statistics

The major fire problem in the United States appears to be in residential structures. Most of fires start in areas that are compartmentalized, giving occupants and firefighters a longer period of time to work, and to safely exit the structure than would be the case if the fires started in an unprotected area or a concealed space.

The non-residential fire problem is decreasing and great improvement in life-safety has been shown. When compared to residential fires, non-residential fires cause more injuries to firefighters—probably due to the greater number of hazards encountered in these fires.

It is interesting to note that in most of the sprinkler activations detailed, one head usually controlled the fire. It is also interesting that the room of origin for these fires was consistent with those shown in the statistics for residences and apartments. Generally, the room of origin is in an area that is compartmentalized and a primary living area, such as the kitchen, bedroom or living room. This further suggests that the focus ought to be on protected lightweight building components.

Sprinklers will be important in the future for diminishing the impact of fires in any type of construction. It is proven that sprinklers reduce property loss and life loss. There is also a strong possibility that sprinklers could reduce firefighter fatalities, since they contain, and even extinguish, fires prior to arrival of the fire department. Sprinklers are currently the most pro-active fire safety approach in building construction.

Regarding firefighter fatalities, **Fire Command** data indicates that 3.8% (45 total) of all firefighter fatalities for the ten-year period from 1980-1989 were due to some type of floor or roof structural collapse. NFPA Fire Analysis and Research Division data for the period from 1979-1988 indicate that 54 fatalities were caused by the roof, floor or ceiling collapse. This represents 4.2% of all firefighter fatalities for this period. These figures include all structural materials types, such as solid-sawn joists, heavy timber trusses, wood trusses, steel trusses, etc.

The **Fire Command** study separated out the categories of non-combustible roof/floor systems, light frame trusses and timber trusses, and found the fatalities to be 0.55%, 0.46%, and 0.42% of total fatalities, respectively. The NFiPA study found that 12 firefighters died in buildings using truss construction, or 0.9% of all fatalities for the time period under study, corroborating the **Fire Command** data.

These data indicate that the number of the lightweight component construction-related firefighter fatalities due to structural collapse is very small. It implies that lightweight component construction has not increased the hazard for firefighters over and above the

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[55] Ibid.

hazard that has always existed on the fire ground. Performing a risk assessment would be helpful in analyzing this data further. Yet, one firefighter fatality due to structural member failure is one too many. The statistics don't provide insight into how many hjeart attack fatalities were triggered by the shock of collapse conditions. The statistics are also not detailed enough to provide more information about the contribution lighteight building construction makes to fire-related problems and fatalities.

Chapter 4: Fire Performance-Related Testing of Structural Assemblies

As noted in previous chapters, a large amount of information is available on the fire performance of lightweight component assemblies, including opinions, experience, and standard and non-standard tests. While there has been criticism of the adequacy of a number of tests to assess fire performance (see **Chapter 2: Literature Review**), only standardized fire testing permits an accurate evaluation of comparative performance.

4.1 Types of Tests Performed

ASTM E119, "Standard Methods of Fire Tests for Building Construction and Materials," is the primary standard used to measure the fire performance of floor/ceiling, roof/ceiling, and wall assemblies and columns, and is the test recognized and accepted by most building codes. The key elements of the ASTM E119 test are[1]:

- Each test follows the ASTM E119 standard time/temperature curve.
- The assembly to be tested is fully instrumented with at least 9 thermocouples, which in the case of roofs, floors and walls are located on the unexposed surface of the specimen. The instrumented locations are specified to provide measurement of thermal transmission through the assembly. This is one of three criteria used to determine the assembly's fire resistance rating.
- The test specimen is intended to represent the construction for which classification is desired. Each specimen is conditioned prior to testing so that its temperature and moisture content is representative of the assembly in its actual environment.
- The area of the assembly exposed to fire is defined. The area for walls and partitions shall not be less than 100 $ft.^2$, and the area for floors and roofs shall not be less than 180 $ft.^2$.
- The load applied to the test specimen shall be a constant superimposed load that, unless specified by the sponsor, applies the maximum allowable design stresses pursuant to recognized structural design criteria.
- The conditions of acceptance for a particular assembly classification are:
  - The specimen shall sustain the applied design load for the duration of the test.
  - At no time during the test duration shall cotton waste be ignited while placed over the unexposed surface.
  - The average temperature rise on the unexposed surface shall not increase more than 250° F (139° C) above its ambient temperature.

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[1] **ASTM Fire Test Standards**, sponsored by ASTM Committee E 5 on Fire Standards, 2nd Edition, 1988, pp. 43-69

- The temperature at any single thermocouple shall not rise more than 325° F (181° C) above the initial temperature.
- For steel structural members, the temperature of the steel shall not exceed 1300° F at any location during the classification period.
- The average temperature on the steel specimens shall at no time exceed 1100° F.
- In concrete specimens with tension steel, the temperature shall not exceed 800° F for cold-drawn pre-stressing steel, or 1100° F for reinforcing steel.
- In wall assemblies, the test specimen is also subject to a hose stream test. For 1-hour assemblies, the water is applied at 30 psi for one minute to simulate specimen stability under suppression activities.

• The rating periods are typically expressed in terms of time, i.e. 45 min., 1-hour, 2-hour, etc.

Small-scale tests are often performed using the ASTM E119 time/temperature curve to evaluate the performance of a combination of materials prior to testing in the large-scale furnace. In some cases, the small-scale test facilities have the capability of applying load. In others, it is mainly a means of evaluating the temperature profiles developed in the small-scale furnace, to predict their performance in large-scale tests.

The ASTM E119 test was developed under the consensus standards development procedures of ASTM, and can be used to satisfactorily *compare* performance of materials under standardized test conditions. Several other tests have been performed on assemblies under what would be termed 'ad hoc' conditions.[2] When a test is conducted using this type of procedure, it is very difficult to compare the performance of one assembly to another.

In other cases, 'ad hoc' testing is done using parts of the ASTM E119 standard. Often, the time/temperature curve is used while the load and assembly size are varied. These tests are usually performed primarily for gathering information, not for model code acceptance.

There is also a standard guide for room fire experiments, ASTM E603. This guide covers full-scale compartment fire experiments that are designed to evaluate the fire characteristics of materials, products or systems under actual fire conditions. This set of procedures is only a guide for room test procedures, experiment design, and result interpretation.[3] At this time there is no accepted ASTM standard test procedure for room fire tests that can be used to evaluate the performance characteristics of roof, floor or wall structural elements.

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[2] Instances when testing was done using non-standardized procedures will be denoted in the summary of the reports.

[3] Taken from the scope statement of ASTM E603.

A variety of international test standards follow ASTM E119 test procedures quite closely, if not exactly. These will also be briefly discussed in this chapter.

4.2 Key Testing Characteristics

The following characteristics are typically measured during testing to allow for analysis and comparison of test results:

4.2.1 Load

Applied load is extremely important in the performance of a structural assembly. In order to make accurate comparisons between assemblies, the impact of the superimposed loads must be equivalent. Generally, this is achieved by applying a load that reaches the maximum allowable design stress on the assembly. Also, an attempt is usually made to maximize the stress on what is considered to be the most critical component of the assembly. The test design attempts to maximize other stresses on the assembly to as near allowable design stress as is feasible from an engineering perspective. If this is not done, the reduced loading must be given special consideration when comparing the performance of assembly types.

4.2.2 Fire Exposures

There has been considerable discussion and debate on the best fire exposure to use for assessment of structural members in a fire. Most standardized tests employ a fire of increasing temperature over time, referred to as time/temperature curves. The current ASTM E119 time/temperature curve is shown in Figure 27. For example, the time/temperature curves in Australian Standard 1530 and British Standard BS 476 closely resemble this curve.

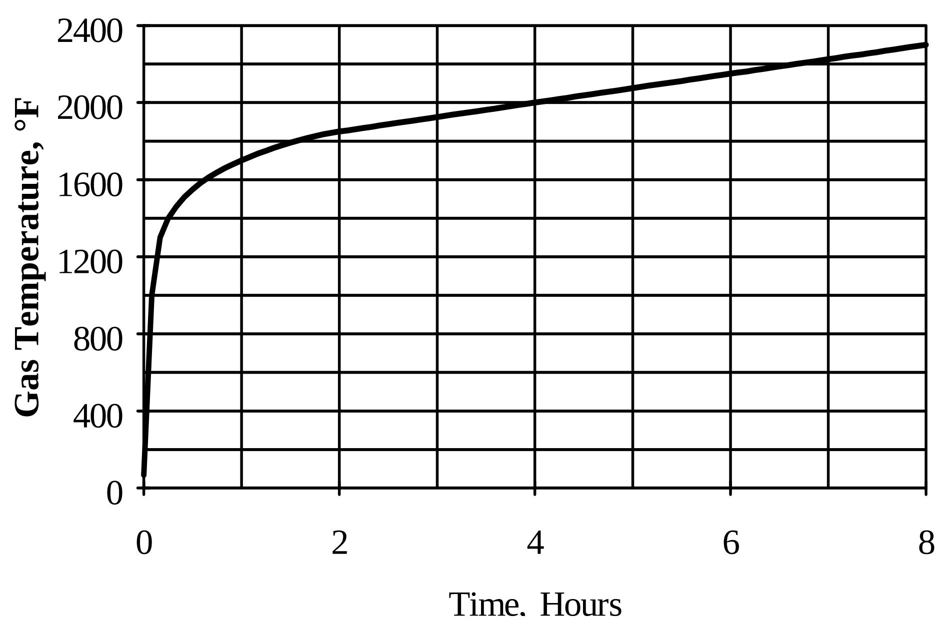

Figure 27. ASTM E119 Time/Temperature Curve

A figure that shows the severity of the ASTM E119 time/temperature in terms of material properties has been prepared and is shown in Figure 28.

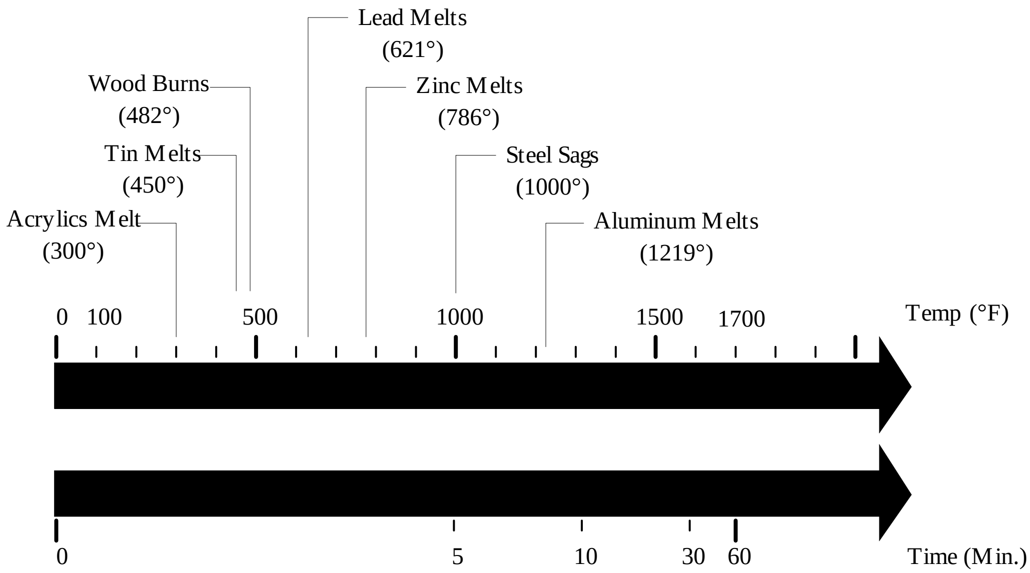

Figure 28. ASTM E119 Standard Fire Exposure [4]

Use of other fire exposures has been suggested. Frank Brannigan and others have been proponents of the NBS-developed time/temperature curve (Figure 29) as more accurately reflecting a contents fire for a residential structure.

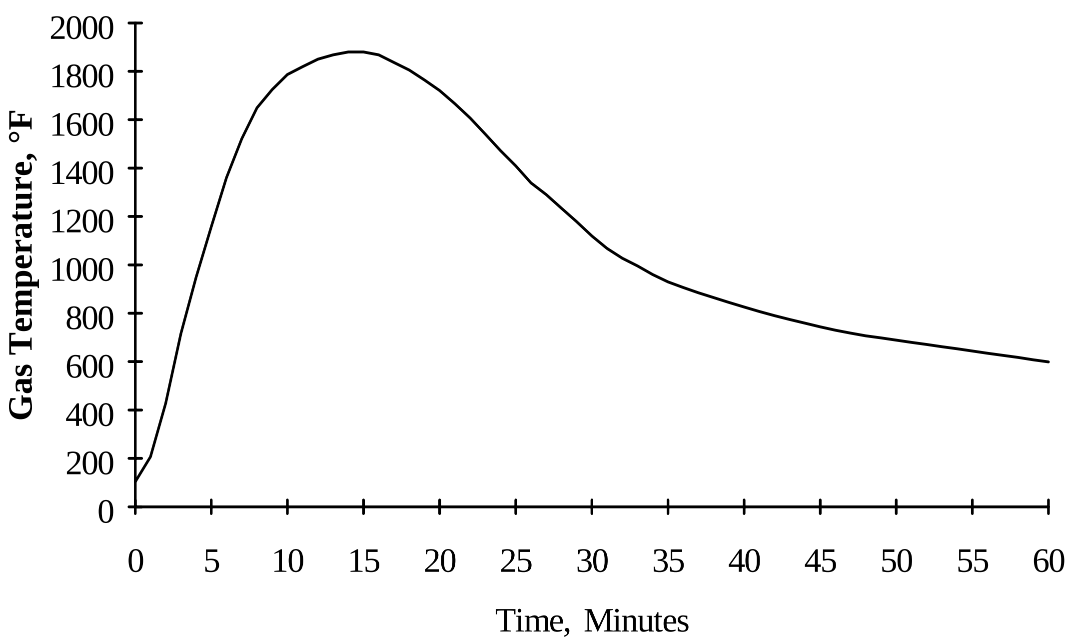

Figure 29. NBS Room Fire Test 9 Time/Temperature Curve[5]

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[4] Truswal Systems Corporation, "What About Wood Trusses and Fire?" Copyright 1984.

Australian researchers use a time temperature curve developed by Rodack and Ingberg (Figure 30) for typical fires in a residential building.

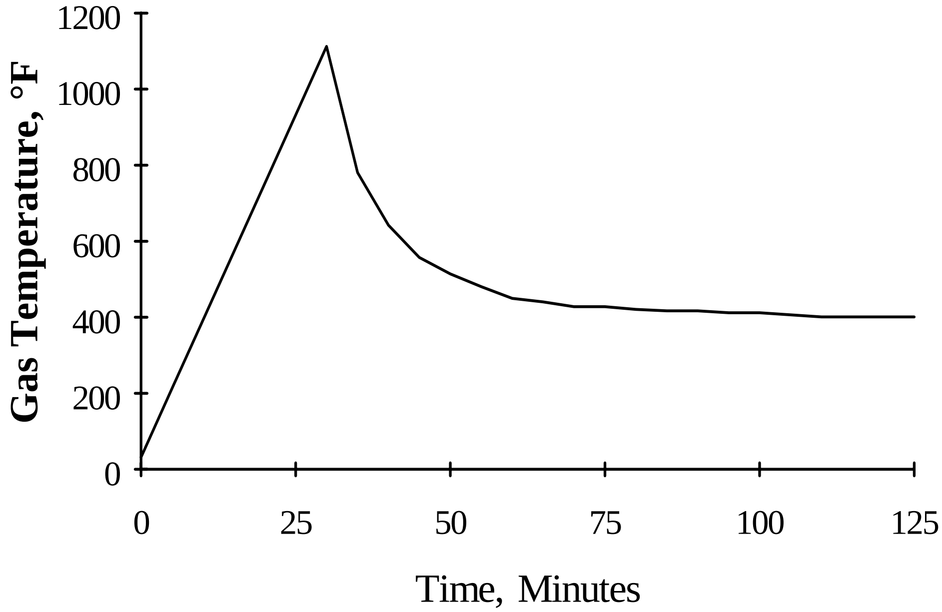

Figure 30. Rodack and Ingberg Time/Temperature curve for Residential Buildings[6]

In ad hoc testing, Captain John Mittendorf of the Los Angeles Fire Department used four gallons of paint thinner and sawn pallets as a representative fire exposure.[7] Similarly, the Illinois Fire Service Institute used four gallons of diesel fuel and one gallon of gasoline in five gallons of water as a fire exposure.[8]

Fire tests conducted to evaluate sprinklers and full-scale rooms often use a combustible crib, a specified combustible commodity (e.g., paper or furnishings), or a propane-burner substitute as the fire source. These fire sources are standardized as much as possible.

In order to make realistic comparisons between assemblies, the fire source must be repeatable. The time/temperature curve used most often is from ASTM E119. Therefore, this method provides most of the available test data. Other time/temperature curves have been used primarily for experimental purposes.

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[5] Brannigan, F.L., "Are Wood Trusses Good for Your Health?" Fire Engineering, June 1988.

[6] Leicester, T.H., Seath, C.A., and Phau L., "The Fire Resistance of Metal Connectors," Proceedings of the 19th Forest Products Research Conference. Melbourne, Australia, November 12-16, 1977.

[7] Mittendorf, J., "Lightweight Constructions Tests Opens Fire Service Eyes to Special Hazards," Western Fire Journal, January, 1982.

[8] Straseske, J. and Weber, C., "Testing Floor Systems," Fire Command, June, 1988.

4.2.3 Furnace Pressure

Adjustment of furnace gas pressures is sometimes used to simulate the pressures occurring in actual and large-scale fires. Because of the significant effect on pressure from items such as openings and their location, there is little consensus as to the establishment of specific pressure levels for fire tests. The ASTM E119 standard does not provide guidance for pressures to be applied in an assembly test. The more positive the pressure on a test assembly, the quicker the fire may penetrate protective membranes or sheathing materials. Generally in the United States, floor/ceiling assemblies are tested with a mild amount of positive pressure due to the natural buoyancy occurring in the test furnace. In Europe, assemblies are tested under a specified amount of positive pressure.

4.2.4 Fuel Load

In ASTM E119 testing, natural gas is the fuel typically used. Generally, the flow of fuel to the burners is monitored to ensure uniform heat throughout the furnace.

When a solid fuel is used to evaluate the fire performance of a system, it is far more difficult to develop fires of consistent quality. The only way to accurately evaluate assemblies comparatively is to use identical fuel loads and types of fuel.

4.2.5 Restraint

Restraint addresses the ability or inability of a structural member to expand under fire conditions. An assembly is restrained if the effects of fire are resisted by forces external to the element. An assembly is unrestrained if the structural element is free to expand and rotate at its supports. In general, steel and concrete systems are tested as restrained members due to the expansion characteristics of steel, and steel reinforcement. Because of its thermal stability, wood is tested as an unrestrained system. Determining if elements are tested under restrained or unrestrained conditions is an area where engineering judgment must be used, since it is not specified in ASTM E119.

4.2.6 Ventilation

ASTM E119 has no direct provision for ventilation. The air required for combustion of components in a fire test assembly is controlled by the need to provide a standard fire exposure. The National Bureau of Standards (NBS) ran several tests using high and low amounts of excess air. These results are reported in later sections of this study. These tests must be distinguished from standard tests where oxygen for combustion must be controlled to maintain the standard time/temperature exposure.

4.2.7 Deflection

Deflection measurements are typically made in assembly testing, but are not required by the ASTM E119 standard. This information is useful in determining deterioration of strength and imminent collapse. It is also useful in comparing deflection performance between assemblies. Deflection and rate of deflection are measures of a system's plasticity under fire conditions and the potential a system has for collapsing without much warning.

4.2.8 Size

In most test standards, the size of the assembly is specified. Size is important—specifically with regard to the applied load. The smaller the specimen size, the higher the applied load will need to be to provide the maximum allowable stress on the member. Size also influences stresses that are critical from a design perspective. In short roof or floor assembly specimens, shear becomes critical; in longer specimens, bending moment, the moment of inertia, and modulus of elasticity (MOE—the stiffness, or ability to resist deflection) become critical.

4.2.9 Test Duration

The duration of the test is usually determined based on some end-point criterion. The end point could be a specific temperature level, load carrying capacity, deflection performance, time period, etc.

4.2.10 Element Tested

Tests are generally performed on either single elements or a combination of elements in an assembly. Single element tests usually produce times of the shortest duration. As an explanation, Dr. Tibor Harmathy has established a number of rules for fire endurance calculations.[9] He states that elements tested as part of an assembly will always perform better than elements tested singly. Therefore, if one has test results for a variety of structural elements, elements can be substituted for one another if the element being substituted has a better fire endurance performance under standardized conditions.

4.3 Test Results

In addition to statistics (as discussed in **Chapter 3**), the base of knowledge on the performance of lightweight building components exists in the form of test data. To aid in providing background, report summaries obtained from the literature search of pertinent tests have been prepared covering the following areas:

- unsheathed assemblies
- single membrane protected assemblies
- connections
- "Operation Breakthrough" assemblies

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[9] Harmathy, T.Z., "Ten Rules of Fire Endurance Ratings," **Fire Technology**, 1(2), pp. 93-102, May, 1985.

- assemblies protected with coatings
- sprinklered assemblies

All available data from these tests have been summarized. Comments regarding the individual tests are given at the end of each summary. Analysis of the testing and test data can be found in **Chapter 7: Discussion**. Conclusions are found at the end of each section, as well as in **Chapter 8: Conclusions and Recommendations**.

It should be noted that with respect to concerns expressed about lightweight component construction, the mode of assembly failure is critical. Non-structural assembly failure (i.e., temperature rise or flamethrough of the sheathing), while very important, does not raise the same concerns with respect to this study, and is not the subject of this report.

Chapter 4-1: Fire Endurance Performance of Unsheathed Assemblies

4-1.1 Report: Lightweight Construction Tests Open Fire Service Eyes to Special Hazards

Author: J. Mittendorf

Sponsor: Los Angeles City Fire Department

Date: May, 1981

Basic Test Description: Metal plate connected (MPC) trusses, wood I-beams (also known as wooden I-joists), open pin-end connected steel web (PECSW) construction, and panelized construction were subjected to fire conditions. The general concept of the test was to utilize typical construction, and observe and record behavioral characteristics.

Test Methods Used: The test specimens were constructed to represent actual field conditions. Trusses used the correct on center spacing; 1/2 in., 3/8 in. or 3/4 in. CDX plywood decking; and were hung or supported as they would be in normal installations. The span of the construction was limited to the size of the donated products. Each test fire was generated from four gallons of paint thinner and sawn pallets. The fire exposure for each test was believed to be approximately equal. No live load was imposed on any of the structures. The test time began at ignition of the thinner and pallets. A time limit of 6 min. per test was used.

Report Observations:

Structural Member	Span (ft.)	Spacing (in. o.c.)	Sheathing Material	Failure Time (min:sec)
Wood I-beams	12	32	1/2" CDX ply.	1:20
PECSW construction	20	24	1/2" CDX ply.	3:20
MPC Truss floor system*	30	16	3/4" CDX ply.	5:00
MPC Truss roof system*	Unknown	32	1/2" & 5/8" CDX ply.	6:00
8 x 8' panel. sys., 2x4 joists	8	24	1/2" CDX ply. & 1 x 6" sheath.	did not fail

* Penetration depth of gusset plate teeth = 3/8 in.

Table 10. Non-Standardized Test Results.

Report Summary: Testing revealed the following:

I-Joists: Once the 3/8 in. web burned and weakened, the entire structure weakened and failed.

PECSW Construction: The weak point of this construction was the junction of the 2 x 4 chord, steel tube webbing and pins. After collapse of the test sample, it was evident that each junction point that had significant char failed.

Metal Gusset Plate Trusses: The early collapse of this test sample was caused by two factors: 1) It was found that once the 2 x 4s charred to a depth of 0.25 in., the gusset plates pulled out; 2) Because the 2 x 4 ends were butted together and held by gusset plates, there was no structural integrity once the gusset plate was pulled free. This allowed the 2 x 4s to separate, causing failure. When metal gusset plates were exposed to fire, the following factors contributed to failure:

- The amount of load or stress imposed on a joint.
- The ability of the plate to conduct heat to the prongs, which causes the wood to expand, and lose its grip on the gusset plate prongs.
- The depth and penetration of the prongs.

When 3/8-in. prongs are used, once the wood is charred 1/4 in., there is only 1/8 in. of wood left creating a friction bond.

Metal Plate Connected Roof Trusses: This truss used a continuous 2 x 4 bottom chord, which was instrumental in the truss' ability to resist failure. Upon close inspection, it was evident that this construction was about to fail when the test was stopped at six minutes.

Panelized: This test was used to compare the difference between roof decking used today and that used 20 years ago. It is obvious that there is no comparison. After extinguishing the fire, it was still possible to walk on the 1 x 6 sheathing, whereas the plywood sheathing of previous tests had been destroyed.

Report Conclusions: The need for each firefighter to become familiar with new developments that will effect job performance and job safety, and pre-fire planning cannot be emphasized enough. For the firefighter, this means a reduction in time to work on buildings which were built with lightweight construction. Although most of the test samples were smaller than what would be found in practical applications, each test resulted in early failure of the construction. Each test was relatively basic; however, each produced similar results when compared with recent fires that have involved this type of construction. Consider what today's firefighter will encounter when faced with normal spans, air conditioning or heating equipment, and several truckmen on the roof.

Comments: THIS WAS THE INITIAL TESTING DONE IN AN ATTEMPT TO DETERMINE THE RELATIVE PERFORMANCE OF LIGHTWEIGHT COMPONENT SYSTEMS. WITHOUT STANDARDIZATION (I.E.), USING IDENTICAL SPAN, SPACING CONDITIONS AND SHEATHING MATERIALS), IT IS DIFFICULT TO MAKE COMPARISONS BETWEEN THE CONSTRUCTINO TYPE TESTED. THIS IS DUE PRIMARILY TO THE FACT THAT THERE ARE NOT EQUIVALENT STRESSES BEING PLACED ON THE MEMBERS TESTED. SPECIFIC DETAILS ON THE STRUCTURAL MEMBERS SUCH AS FLANGE SIZE AND DEPTH OF I-JOISTS WOULD BE EXTREMELY

VALUABLE IN EVALUATDING THESE TESTS AS WELL. THIS INFORMATION WAS NOT AVAILABLE IN THE REPORT.

THE REPORT ALSO STATES THAT THE FAILURE WAS EARLY, BUT DOES NOT DEFINE "EARLY". THE RELATIVE MEANING BEHIND THIS NEEDS TO BE CLEARER.

4-1.2 Report: Testing Floor Systems

Authors: J. Straseske and C. Weber

Sponsor: Illinois Fire Service Institute at the University of Illinois

Date: Fall, 1986

Test Methods Used: The floor systems used for demonstrations were:

1) Conventional 2 x 10 joists on 16 in. centers.
2) Wood I-beams on 24 in. centers.
3) Open-web trusses with wood members and gusset plates on 24 in. centers.
4) Open-web trusses with a stamped out steel webs on 24 in. centers.
5) Open-webbed trusses with a wooden top and bottom chord and pipe web members on 24 in. centers.

All decks were identical in size, manner of loading and ignition source. The decks were built 8 x 9 ft., and were set up on 8 in. concrete blocks, three layers high. Block foundations enclosed each system on three sides. Each deck was placed on a 2 x 6 sill plate mounted on top of the blocks. The 2 x 10 system used a 2 x 10 box sill. The open-webbed truss systems were enclosed on the outside perimeter of the deck by 3/4 in. plywood to enclose the box sill. All deck systems were sheathed with 3/4 in. tongue-and-groove waferboard that was nailed down with 8 penny, coated nails. A live load of 31 psf, consisting of concrete blocks, was distributed across each deck. The fuel for the fire was contained in cut-off 55 gal. barrels approximately 12 in. high. The ignition fuel source was 4 gal. of diesel fuel, 1 gal. of gasoline, and 5 gal. of water.

Report Observations: The 2 x 10 platform began to sag at 8 min., and burned through the sheathing at 9 min. No further significant damage occurred, and the fire was extinguished at 13 min. The 2 x 10 system continued to carry its load after the 13 min. burn. The system gave ample warning that a structural problem was developing: it sagged, but the system did not fail entirely.

At 4 min., 40 sec., the wooden I-beam platform failed completely. There was no sagging or warning noises to indicate a structural problem. The system carried the load until failure. The failure of the wood I-beam system to sag prevents firefighters from determining if the building is in structural trouble.

The metal plate connected wood truss system began sagging at approximately 8 min., and burned through the sheathing at 9 min. The sagging of the floor was very evident, but the system continued to carry the load until the fire was extinguished at 15 min., 45 sec. By sagging, this system gave a definite indication of structural problems.

The metal web wood truss began sagging at 6 min. Most notable was that the metal web failed to carry the load. As the web failed, the top and bottom chords came together. The fire was extinguished at 7 min., 30 sec., when the fire burned through the sheathing.

Burn through of the sheathing of the pin-end steel webbed wood trusses occurred at 6 min., 50 sec. At burn through there was no noticeable sag. At approximately 8 min., parts of the bottom chord were hanging down into the fire. At 9 min., 45 sec., the system failed without any warning or sagging.

A summary of these tests results is shown in Table 11.

Structural Member	**Spacing**	**Assemb. Rating (min:sec)**	**Structural Failure (min:sec)**	**Loading (psf)**
2 x 10	16 in. o.c.	9:00[1]	> 13:00	31.0
I-joist	24 in. o.c.	4:40[1]	4:40	31.0
MPCT[2]	24 in. o.c.	9:00[1]	15:45	31.0
MPSWT[2]	24 in. o.c.	7:30[1]	N/A	31.0
TJL	24 in. o.c.	6:50[1]	9:45	31.0

[1] Assembly rating is due to deck burn through.
[2] MPCT = Metal Plate Connected Truss; MPSWT = Metal Plate Steel Web Truss; TJL = Trus Joist L-Series Truss; TPSB = Truss Plate Spliced Beam; F_b = fiber bending stress.

Table 11. Non-Standardized Test Results.

Report Summary: These test results tend to show that some of the truss systems have inherent strength. They also show that open web trusses allow for rapid lateral spread of the fire, and that some systems give no warning prior to collapse. The following thoughts can be drawn from this testing:

- Pre-plan all new construction and any remodeling using lightweight components.
- Modify fire department tactics to open up concealed spaces very quickly.
- Push for modification for building codes to control the amount of square footage that can be built with these lightweight components, without firestops.
- Maintain records of all buildings using lightweight components.
- Be aware of the time factor—always ask, "How long has the fire been burning?"
- Remember—some floor systems give no warning prior to collapse.

The times stated in this article from ignition to collapse are those found in these test fires. They should not be taken as a guarantee that various floor systems will last as long in every fire. The time should serve only as information when making decisions about fire suppression operations.

Comments: MAKING VALID COMPARISONS BETWEEN THESE TESTS IS VERY DIFFICULT. THESE ARE MORE STANDARDIZED THAN THE TESTS DESCRIBED ABOVE IN THAT ALL SPANCS ARE EQUIVALENT AND THE LOAD IS CONSISTENT. THE KEY TO MAKING A VALID COMPARISON, HOWEVER, IS IN HAVING AN EQUIVALENT STRESS BASIS, WHICH IS NOT THE CASE HERE. WHEN COMPARING THESE TESTS TO THE **Section 4-1.1** TESTS, IT IS CLEAR THE SPAN INFLUENCED THE TIME TO FAILURE. AS WITH THE **Section 4-1.1** TESTS, MORE DETAIL IS NEEDED IN THE TEST REPORT ON THE SPECIFICS OF THE STRUCTURAL MEMBERS USED IN THIS TESTING. WITHOUT THIS DETAIL, DEEPER ANALYSIS OF THESE TESTS IS NOT POSSIBLE. IT IS INTERESTING TO NOTE THAT THE METAL PLATE CONNECTED TRUSS PERFORMANCE IS EQUIVALENT TO THE SOLID-SAWN JOIST PERFORMANCE IN THESE TESTS. ALSO, THE PERFORMANCE TIMES INCREASED WHEN COMPARED TO THE **Section 4-1.1** TESTS. THIS POINTS OUT THE IMPORTANCE OF MAKING COMPARISONS BETWEEN TESTS THAT HAVE STANDARDIZED TEST PROCEDURE. WITHOUT THIS, VALDI COMPARISONS CAN NOT BE MADE.

4-1.3 Report: Comparative Fire Endurance Tests of Unprotected Engineered Wood Component Assemblies.

Authors: Proprietary

Sponsor: Proprietary

Date: April, 1988

Basic Test Description: The members tested were: 9.5 in.) TJI 25 series joists (a Truss Joist Corporation product, 10 in. metal plate connected trusses, 10.75 in. space joist- or metal web-trusses, and 2 x 10 dimensional lumber. The members were tested as unsheathed, with single units subjected to a 500 lb. load at center span for the test duration. Modifications were made to the various members so that the critical components were stressed to approximately 30% of the allowable design load under the given load and span conditions. In addition, holes were cut in the I-joist web conforming to building code approval. Holes and notches were removed from the solid-sawn joists in conformance with model building codes. Each test specimen was 8 ft., 1 in. long.

Test Methods Used: The fire tests were performed using ASTM E119-83 as a guide. The I-joists were stressed to approximately 30% of their allowable moment- and shear capacity. The solid-sawn joists were stressed to approximately 40% of their allowable moment capacity, and 30% of their allowable shear capacity. The truss plates were sized to achieve a truss design that would be stressed to approximately 30% of the allowable load capacity for both the metal plate connected- and space-joist trusses. Each member type was modified in order to approach a worst-case scenario under test conditions. A hole representing 40% of the allowable hole size was removed from the web at center span of each I-joist. In addition, all pre-cut knock-out holes were removed. Notches and holes were cut into the solid-sawn joists in accordance with building code criteria. The maximum allowable holes and notches were used. The trusses were designed with compression- and tension chord splices located at one of the quarter points of the chase opening. Maximum chase openings were used in both trusses.

Report Observations: The two wood I-joist specimens were tested first, and the time/temperature readings were used as a guide for the remaining specimen tests in an effort to produce repeatable results. Ignition of each specimen was targeted for between 2 and 3 min. after furnace ignition. Similar temperatures at corresponding thermocouple and time periods indicated the furnace conditions were fairly consistent for each test. Deflection and time to failure was measured for each test.

Report Summary: The test data indicate that wood I-joists, metal plate connected trusses, and space-joist trusses exhibit similar performance characteristics. The deflection for each was small, until the member temperature reached 1000° F. Deflection increased dramatically after that point. The deflection was slightly greater for the wood I-joist than for the two types of trusses tested. This would suggest that trusses undergo a more gradual relaxation in load carrying capability as they burn, when compared to I-joists. Failure typically began near the 3 min. mark, and was completed by 5 min. in these members.

Six minutes after furnace ignition, the solid-sawn joists exhibited less than 10% of the mid-span deflection observed with other member types. After ten minutes, the joists had deflected only one inch. The joists did not begin to deflect appreciably until the member temperature reached 1000° F. Once this temperature was attained, mid-span deflection increased at an ever-increasing rate. Exactly when failure would have occurred for the solid-sawn joist is unknown.

Based on the results of this study, the following conclusions can be made:

- The wood I-joists, metal plate-connected trusses, and metal web trusses appear to have similar fire endurance capabilities. The fire endurance performance of these products were dependent on their critical components, which are the web for the I-joists and the tension splice for the trusses.
- The 2 x 10 solid-sawn joist performed better than the engineered wood components in these tests.

The results and conclusions of this study must be maintained in the proper context. Due to the limitations of the test facility, it was difficult to control as many of the variables as would have been preferred in order to accurately assess comparative fire endurance performance. Given proper control of these variables, it is felt that more accurate comparisons between member types can be made.

> **Comments:** This is the first series of tests that have attempted to perform comparisons on a standardized, equal-stress basis. These were performed on short-span, single elements and not on an assembly. The small-scale nature of the test facility made it difficult to achieve equivalent loads on critical members. In this case, due to the short span, shear stresses predominated. Ideally in this testing, bending moments or extreme fiber tension stresses

CAUSING FAILURE OF THE ELEMENT WOULD BE PREFERRED. NEVERTHELESS, THIS TESTING SHOWED THAT SOLID-SAWN JOISTS PERFORM BETTER THAN THE LIGHTWEIGHT COMPONENTS TESTED. THESE TESTS PROVIDE ONLY A VERY ROUGH VIEW OF RELATIVE PERFORMANCE DUE TO THE FACT THAT MANY VARIABLES COULD NOT BE CONTROLLED IN THE SMALL-SCALE TEST FURNACE.

4-1.4 Report: ASTM E119-73: Fire Endurance Test on a Floor Assembly (Design FC-209) Consisting of 2 x 10 Wood Joists with a 23/32 in. Plywood Deck and Vinyl Tile Flooring.

Authors: Factory Mutual Research

Sponsor: National Forest Products Association

Date: June 20, 1974

Basic Test Description: The construction contained nominal 2 x 10 wood joists spaced 24 in. on center. The floor consisted of a single layer of 23/32 in. thick plywood underlayment, with a vinyl-asbestos tile flooring. No ceiling membrane was installed. The joists were nominal 2 x 10 Southern Pine #2, S-Dry 1250 fiber bending. The joists—each 13 ft., 6.75 in. long—were fastened to a 2 x 10 header on a 2 x 6 wood plate.

Test Method Used: The test was conducted in accordance with the standard fire test of building construction materials, ASTM E119-73. Before the assembly was subjected to fire exposure, a superimposed live load of 57.4 psf was applied to the floor. The total live and dead load of 62.1 psf was based on the repetitive member fiber stress of 1450 psi in bending and a joist depth of 9.25 in. The clear spans of the joists were 12 ft., 10.75 in. The exposed underside of the floor assembly was subjected to the fire exposure. The temperature in the furnace followed the standard time/temperature curve as measured by 16 thermocouples placed 12 in. below the lower edge of the joist.

Report Observations: Deflection measurements, structural failure of the system, and a number of other visual observations were recorded during the tests.

Report Summary: The floor assembly withstood fire exposure for 13 min., 34 sec. before structural failure occurred. Analysis of the unexposed surface temperature chart indicates that the average temperature at 13 min. of fire exposure was 150° F, while the allowable average temperature was 320° F. The average deflection at failure was 2.83 in.

Comments: This test is one of the first unsheathed tests where the joists are stressed to their maximum allowable design load capacities. Therefore, this assembly can be compared to other unsheathed assemblies that follow the ASTM E119 procedures in total.

4-1.5 Report: ASTM E119 Fire Endurance Test of a Floor Assembly (Design FC-212) Consisting of 2 x 10 Wood Joists with a 23/32 in. Plywood Deck and Nylon Carpet Flooring.

Authors: Factory Mutual Research

Sponsor: National Forest Products Association

Date: July 17, 1974

Basic Test Description: The construction contained nominal 2 x 10 wood joists grade marked Southern Pine Inspection Bureau #2 kiln-dried, 1250 fiber bending stress. The joists were cut to a length of 13 ft., 6.5 in., secured to a 2 x 10 wood header, and fastened to a 2 x 6 wood bearing plate. The joists were spaced 24 in. on center. The floor consisted of a single layer of 23/32 in. thick plywood underlayment with a nylon carpet flooring. No ceiling membrane was installed.

Prior to the assembly being subjected to fire exposure, a superimposed live load of 57.3 psf was applied and maintained throughout the test. The combined live and dead load of 62.4 psf was based on a clear span of 12 ft., 10.5 in. The loading was calculated to stress the joist to a maximum repetitive member design stress of 1450 psi in bending. The underside of the assembly was subjected to fire exposure. The temperature in the furnace followed the standard time/temperature curve as measured by 16 thermocouples which were placed 12 in. below the joists.

Report Observations: Deflection measurements, time to failure, and other visual observations of the tests assembly were recorded for this test.

Report Summary: The floor assembly withstood fire exposure for 12 min., 6 sec. before structural failure occurred. Analysis of unexposed surface temperatures indicate that the maximum individual temperature recorded during the test was 103° F, while the allowable individual limiting temperature was 398° F. The average deflection at failure was 3.58 in.

Comments: When compared with the tests in **Section 4-1.4**, this test identified the effect of floor coverings, carpet and vinyl on fire performance. This aspect appears to have little bearing on test results. The test performance range for the similar unsheathed 2 x 10 joist tests is 12 to 14 minutes. Additional testing could to be done to determine how wide this range actually is.

4-1.6 Report: Fire Endurance Test of Unprotected Wood Floor Constructions For Single-Family Residences, NBS 421346.

Author: B.C. Son

Sponsor: United States Department of Housing and Urban Development

Date: May 10, 1971

Basic Test Description: Part of a series of fire tests. In two tests, numbers 2 and 4, the 2 x 10 joists were Douglas fir, which was assumed to have a stress level of 1050 psi in bending. The joists were spaced 16 in. on center, with a span of 13.5 ft. A load of 63.7 psf was calculated to produce an extreme fiber bending stress of 1050 psi in the joists, and was applied to the floor through four hydraulic jacks. One half of the specimen consisted of two layers of 1/2 in. plywood with no covering, while the other half consisted of two layers of 1/2 in. plywood with nylon 501 carpet over a hair pad underlayment.

Tests 9 and 10 consisted of 2 x 8 Douglas fir joists spaced 16 in. on center. The applied live load was reduced to 20 psf, which resulted in a 21 psf total load. This represented approximately 40% of the working stress of the joists. Two flooring systems were also applied to this test. One consisted of a layer of 1/2 in. thick plywood with a square-edged joint. The plywood was placed leaving a 1/16 in. joint spacing. The joint was protected by nominal 2 x 3 in. blocking. The other area consisted of a 5/8 in. thick tongue-and-groove plywood on all four edges.

Test Methods Used: The load was applied 8 min. before the test started, and was distributed to approximate a uniform load. The average temperature inside the furnace was measured by 12 protected thermocouples, and followed the ASTM E119-69 time/temperature curve by automatic control of the gas flow to the burners.

Report Observations: Smoke development measurements, deflection measurements, time to failure, and other visual observations were recorded. In the 2 x 10 test, load failure occurred at 11 min., 38 sec. On the plywood side only, the flamethrough time was 13 min., 30 sec., and the unexposed side temperature failure time was greater than 15 min. On the side with the double-layer plywood and carpet, neither the flamethrough time, nor the unexposed side temperature failure time was reached. The deflection at failure was 2.7 in. for the side without carpet, and 3.3 in. for the side with carpet.

The 2 x 8 test structurally failed at 13 min. The 5/8 in. tongue-and-groove plywood had flamethrough at 11 min., 50 sec., and unexposed side temperature failure at 10 min. The 1/2 in. spaced plywood with 2 x 3 in. end blocking had flamethrough at 11 min., and unexposed side temperature failure at 9 min. The deflection of the 1/2 in. plywood side was 7 in., and the 5/8 in. plywood side was 12 in., at failure.

Report Summary: Bare wood floor constructions conforming to FHA minimum property standards are able to marginally meet a fire endurance time requirement of 10 min. The addition of a separate finish floor should increase the fire endurance time, depending on its additional thermal resistance. This is estimated to be approximately 30 sec. for 1/8 in. vinyl asbestos tile to approximately 10 min. for carpeting over a hair pad.

> **Comments:** This appears to be the first test done on unsheathed assemblies to determine whether a typical floor system (of that time) could meet the HUD 10-minute exposed floor fire endurance requirements. The joists selected were Douglas fir, which were assumed to have a stress level of 1050 lb./in^2. A maximum design load was applied to the assembly based on this assumption. There is the possibility that the joists used were not at their full design allowable fiber bending stress. These tests add to the knowledge of the performance of 2 x 10 joists and the importance of sheathing to protect against flamethrough. The failure of 11 min., 38 sec. increases the data needed to define the performance range of 2 x 10's.

4-1.7 **Report:** Replicate Fire Endurance Tests on Unprotected Wood Joist Floor Assembly

Authors: R.H. White, E.L. Schaffer, and F.E. Woeste

Sponsor: Forest Products Laboratory

Date: March, 1983

Basic Test Description: Nominal 2 x 10 Douglas fir dimension lumber, 14 ft. long, was used in the tests. The testing consisted of eleven 14 x 18 ft. unsheathed joist floors. Five floors supported a maximum floor load of 79.2 psf (100% of maximum design load based on fiber bending stress, per the test report). Six other floors supported a 11.35 psf (14.3% of maximum design load based on fiber bending stress, per the test report) live load floor that is more typical of the actual loading encountered in residences. Plywood, 23/32 in. thick, was used as sheathing. Fourteen joists attached to headers were used to construct the 14 x 18 ft. frame. The joists spanned 13 ft., 10.5 in., and were spaced 16 in. on center.

Test Methods Used: The standard ASTM E119 time/temperature curve was followed for each floor. Gas burners within the furnace provided the standard fire exposure to the test specimen.

Report Observations: Thermocouples recording the temperature within the furnace and on the test specimens, the atmospheric pressure within the furnace, deflection of the floor, and other visual observations were recorded.

Report Summary: For the six floors loaded to 11.35 psf, the mean time for initial joist failure was 17 min., 54 sec., with a coefficient of variation of 3.7%. The mean time to second joist failure was 18 min., 6 sec., and the mean time to third joist failure was 18 min., 24 sec. For the five floors loaded to 79.2 psf, the mean time for initial joist failure was 6 min., 30 sec. with a coefficient of variation of 11.6%. The mean time for second joist failure occurred at 6 min., 42 sec. and third joist failure occurred at 7 min., 6 sec.

The average deflection of joists loaded to 79.2 psf was roughly 4.05 in. at failure. The average deflection of joists loaded to 11.35 psf was roughly 1.7 in. at failure.

> **Comments:** It is obvious from this testing that the deflection of an assembly at failure is dependent on the loading on the floor—the greater the loading, the more likely deflection will be observable under fire endurance performance conditions. In this testing, loading the floors to the maximum allowable design load of 79.2 psf resulted in an initial joist failure at 6 min., 30 sec. Three joists failed within 7 min. This shows the load sharing effects of floor systems that have member spacing less than 24 in. on center. If it is assumed that failure time for the assembly is the time for the first joist to fail, then at the maximum load of 79.2 psf, the average (mean) failure time was 6 min., 30 sec. However, this would be misleading because of the redundancy provided by the system. As a result, the entire assembly will fail somewhat later than the 6 min., 30 sec. At a little over 7 min., however, three joists had failed. Under the lighter load of 11.35 psf, which is similar to the average live load found in domestic dwellings from three surveys[1], the joist failure time increased to approximately 18 min. It is also interesting to note that the deflection decreased dramatically for joists tested under the 11.35 psf live load, when compared to full load. This study also made an estimate of the time to failure for a joist system under a 40 psf load. This time to failure was interpolated to be approximately 13 min. These data clearly show the effect load has on fire endurance and the time it takes for visible deformation of the assembly to occur during a fire. We also learn that the range of 2 x 10 performance expands to 6 to 7 min. under maximum allowable design loads.
>
> There is a question with respect to the application of the design load and the adequacy of the test apparatus that was used for this test.

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[1] Carmen, 1969; Corotis and Doshi, 1977; and Issen, 1980

THE 79.2 PSF APPLIED LOAD MAY ACTUALLY BE IN EXCESS OF THE MAXIMUM ALLOWABLE DESIGN LOAD, DUE TO THESE FACTORS. GIVEN THIS, THE 2 X 10 FIRE PERFORMANCE RANGE WOULD NOT EXPAND DOWN TO THE 6 TO 7 MINUTE RANGE, BUT WOULD BE HIGHER THAN THIS. THIS DATA SHOULD NOT BE VIEWED AS RELIABLE, BUT RATHER FOR GENERAL INFORMATION PURPOSES ONLY. THE LOWER RANGE OF THE FIRE ENDURANCE PERFORMANCE OF 2 X 10S CANNOT BE PREDICTED USING THIS DATA.

4-1.8 Report: A Floor-Ceiling Assembly Consisting of Wood Trusses with a Plywood Floor. (Design FC-250)

Author: Factory Mutual Research

Sponsor: Truss Plate Institute

Date: May 10, 1977

Basic Test Description: The floor assembly consisted of 12 in. deep floor trusses, made with nominal 2 x 4 wood chords and webs, spaced 24 in. on center. The floor was sheathed with a single layer of 3/4 in. thick plywood. The trusses were exposed from below. The assembly was subjected to a uniformly distributed live load of 55.1 psf, which resulted in a combined live and dead load of 60 psf.

Test Methods Used: The test was conducted in accordance with ASTM E119-76. The temperature of the furnace chamber was measured using sixteen thermocouples 12 in. below the level of the lower chords.

Report Observations: The furnace and surface temperatures, deflections, and other visual observations were recorded during the testing.

Report Summary: The furnace temperatures in this test exceeded the standard time/temperature curve from 5 min. into the test until failure. (No attempt was made to correct the test results due to excessive furnace temperatures, as is allowed by E119 test procedures.) The floor allowed flames to penetrate the unexposed surface at a plywood end joint at 7 min., 30 sec. At 10 min., 12 sec., one of the chains used to move the loading tanks was tight due to the deflection of the floor, resulting in the floor no longer being able to support the applied load. The test was terminated at 14 min., 36 sec. The average deflection at 12 min. was 11.5 in.

Comments: IT IS OBVIOUS FROM THIS REPORT THAT THERE WAS A SUBSTANTIAL AMOUNT OF DEFLECTION IN THIS TRUSS TEST AT FAILURE. THIS WOULD IMPLY THAT TRUSS CONSTRUCTION CAN PROVIDE A WARNING OF IMMINENT COLLAPSE DUE TO THIS DEFORMATION—PARTICULARLY WHEN ONE COMPARES THIS TO THE DEFLECTION PERFORMANCE OF 2 X 10 JOISTS IN **Sections 4-1.4** AND **4-1.5**, WHERE DEFLECTION WAS 3.58 AND 2.83 IN., RESPECTIVELY. IN THIS CASE, THE TRUSSES WERE LOADED TO THEIR FULL

> DESIGN LOAD, AND PERFORMED STRUCTURALLY FOR APPROXIMATELY 10 MIN. THIS IS VERY SIMILAR TO THE 2 X 10 JOIST PERFORMANCE REPORTED PREVIOUSLY. THE ASSEMBLY RATING WAS 7 MIN., 30 SEC. IN THIS TEST DUE TO FLAMETHROUGH AT A PLYWOOD JOINT. THIS COULD BE ELIMINATED BY USING A DOUBLE WOOD FLOOR OR BY ATTACHING A TYPICAL SHEATHING COVERING, SUCH AS CARPETING, TO THE TEST ASSEMBLY.
>
> NOTE, HOWEVER, THAT THESE TRUSSES WERE MANUFACTURED WITHOUT A SPLICE PLATE IN THE BOTTOM CHORD, WHICH WOULD INFLUENCE THE FIRE ENDURANCE PERFORMANCE OF THE TRUSS. MANY FLOOR TRUSSES (TYPICALLY THOSE LESS THAN 20 FT. LONG) ARE MANUFACTURED WITHOUT SPLICE PLATES IN THE BOTTOM CHORD AND THEREFORE COULD BE EXPECTED TO PROVIDE FIRE ENDURANCE SIMILAR TO THAT OF JOIST CONSTRUCTION.

4-1.9 Report: Floor Assembly Consisting of 7.25 in. Deep Steel Joists with 23/32 in Plywood Deck and Vinyl Tile Flooring. (Design FC-208)

Authors: Factory Mutual Research

Sponsor: National Forest Products Association

Date: June 19, 1974

Basic Test Description: Part of a series of tests. This test construction consisted of 7.25 in. deep channel-shaped steel joists made of 16 gauge steel spaced 24 in. on center. The joists were 13 ft., 6.5 in. long, and secured to a nominal 2 x 8 wood header. A single layer of 23/32 in. thick underlayment grade plywood was used as sheathing, and a 1/16 in. thick vinyl tile floor covering was applied. A live load of 65.7 psf was applied to the floor. A total live and dead load of 69.8 psf resulted in a maximum joist bending moment of 34,900 in.-lb. on a clear span of 12 ft., 11 in.

Test Method Used: The tests followed the ASTM E119 standard time/temperature curve as measured by 16 thermocouples, placed 12 in. below the lower flange of the joists.

Report Observations: Temperature of the furnace, temperatures of the unexposed surface of the floor, deflection, and visual observations were recorded for this test.

Report Summary: The floor assembly withstood the fire exposure for 7 min., 24 sec. before flames penetrated the unexposed surface. At 7 min. 30 sec., the floor failed to support the superimposed load. The average deflection at 7 min. was 7 in.

> **Comments:** THIS TESTING PROVIDES DATA FOR COMPARING THE 2 X 10, 12 IN. METAL PLATE CONNECTED TRUSS, AND STEEL CHANNEL-SHAPED JOISTS UNDER ASTM E119 TEST CONDITIONS (SEE FURTHER TESTS BELOW). IN EACH CASE, A SUPERIMPOSED LOAD WAS APPLIED THAT RESULTED IN MAXIMUM BENDING

STRESS ON THE STRUCTURAL MEMBERS. THEREFORE, RELATIVE PERFORMANCE COMPARISONS CAN BE MADE BETWEEN THESE SPECIFIC ASSEMBLIES. NOTE, HOWEVER, THAT THESE REFERENCED TESTS WERE DONE BY DIFFERENT SPONSORS AT DIFFERENT TIMES FOR DIFFERENT REASONS, SO ABSOLUTE COMPARISON MAY NOT BE POSSIBLE.

4-1.10 Report: ASTM E119 Test of a Floor Assembly Consisting of 7.25 in. Deep Steel Joists With 23/32 in. Plywood Deck and Nylon Carpet Flooring (Design FC-211)

Author: Factory Mutual Research

Sponsor: National Forest Products Association

Date: July 16, 1974

Basic Test Description: The assembly consisted of 7.25 in. deep, 16 gauge channel-shaped steel joists, spaced 24 in. on center. These joists were 13 ft., 6.5 in. long and secured to a nominal 2 x 8 wood perimeter joists. A single layer of 23/32 in. thick underlayment was used as a flooring. The plywood deck was covered with sponge rubber waffle pad and a nylon carpet. A superimposed live load of 65.4 psf was applied to the 12 ft., 11 in. clear span joists. The combined live and dead load was 69.8 psf. This resulted in the channel-shaped steel joists being stressed to their maximum design stress of 34,900 in-lb. in bending.

Test Method Used: The assembly was subjected to the conditions of ASTM E119-73.

Report Observations: Furnace temperature, temperature on the unexposed surface, deflection, time to failure, and visual observations were recorded for this test.

Report Summary: At 5 min., 12 sec., the floor assembly failed to support the superimposed load, and the test was terminated. The average deflection of the assembly was 10 in.

Comments: THE ONLY CHANGE TO THIS ASSEMBLY FROM **Section 4-1.9** WAS THE USE OF A PAD AND CARPET COVERING. THE COMMENTS STATED ABOVE APPLY HERE AS WELL. THE RANGE OF STEEL JOIST PERFORMANCE IN THESE TESTS IS 5 TO 7 MIN. ADDITIONAL TESTING IS NEEDED TO FURTHER DEFINE THE FULL BREADTH OF STEEL PERFORMANCE.

4-1.11 Report: Comparative Fire Tests in Wood and Steel Joists

Author: Southwest Research Institute, San Antonio, Texas

Sponsor: National Forest Products Association

Date: 1961

Basic Test Description: The criteria used to develop this test procedure were as follows:

- The test structure should be sufficiently large so that the wood and steel members to be evaluated could be of a size and span representing full-scale roof framing.
- The test enclosure should be such that both roof framing systems could be exposed simultaneously to equivalent fire conditions, and arranged so that each system could react independently.
- A roof load calculated to develop the design capacity of each wood and steel member should be applied.
- The fire exposure should follow the temperatures set forth in the standard ASTM E119 time/temperature curve.

The clear span for both joist systems was 28 ft., and the spacing was 3 ft., 7 in. on center. Clearance beneath the joists was 9 ft., 5 in. The left panel was supported by two 4 x 14 in. solid-sawn wood joists which were designed in accordance with the National Design Specification® for stress grade lumber. The right panel was supported by two 14 in. open web (14S4) steel joists, which were designed in accordance with the manufacturer's recommendations. The roof was designed for a total load of 30 psf. Heat was supplied by six equally spaced, industrial-type gas burners, which were positioned on each side of the structure and directed through ports in the walls.

Test Methods Used: During the test, the flow of gas was regulated to provide uniform test chamber temperatures in accordance with ASTM E119.

Report Observations: Furnace temperature and deflection measurements were recorded during the tests.

Report Summary: The wood and steel joists deflected at the following rates:

Time (min.)	Temperature (°F)	Steel Joist Defl. (in.)	Wood Joist Defl. (in.)
4	900°	1	approx. .16
7	1120°	approx. 3	approx. .33
12	1300°	18*	.5

*This was the limit of the measuring device

Table 12. Wood and Steel Joist Deflection Rates.

At 13 min., the gas was cut off, and the deformation continued to increase until the panel with the steel joists collapsed into the furnace.

The panel with the wood joists supported the full design load during the entire test, and the maximum deflection recorded was 1/2 in. After 13 min. of fire exposure, there remained 80% of the original wood section—undamaged and available to carry load.

The steel joists did not burn, but they failed to support the load under E119 conditions. The wood joists were charred, but continued to support the full design load without appreciable deformation.

Comments: THESE TESTS SHOW THE DIFFERENCE IN PERFORMANCE OF UNPROTECTED STEEL AND UNPROTECTED WOOD. THE UNPROTECTED WOOD IS PROTECTED BY THE CHARRING PROCESS, WHEREAS EXPOSED STEEL RAPIDLY LOSES ITS YIELD STRENGTH AS TEMPERATURES EXCEED 1000° F. THIS IS A GOOD COMPARATIVE TEST, SINCE CONDITIONS BETWEEN STRUCTURAL MEMBERS WAS AS EQUIVALENT AS POSSIBLE, DUE TO THE SPECIALIZED NATURE OF THIS ASSEMBLY. IT IS DIFFICULT TO EXTEND MEANING TO THIS BEYOND THIS SPECIFIC COMPARISON.

4-1.12 Report: Comparative Fire Test of Timber and Steel Beams

Authors: Southwest Research Institute, San Antonio, Texas

Sponsor: National Forest Products Association

Date: Assumed to be 1961.

Basic Test Description: Two beams were evaluated in the same furnace. The left panel was supported by a 16 in. rolled steel beam (designated 16 WF 40), designed for the applied roof load in accordance with American Institute of Steel Construction. The right panel was supported by a 7 x 21 in. glue-laminated timber beam designed in accordance with the National Design Specification® for stress-grade lumber, and the design standards of the American Institute of Timber Construction. Both beams had a clear span of 43 ft., 3 in., and were supplied with 2 in. of camber to offset initial deflection. The roof construction consisted of bulb-tee sections spaced at 32-5/8 in. on center, and attached to the top edges of the beams and exterior walls. gypsum form board, 1/2 in. wide, was placed on the bulb-tees to receive the lightweight concrete deck, which was poured to a depth of 2.5 in. The two sections of the roof deck were entirely separated by a longitudinal joint 2 in. wide, which was covered by a flexible insulating blanket. This allowed each panel to move independently for a vertical distance of 36 in. without loss of heat in the structure. The total design load on the roof consisted of an applied live load equivalent to 30 psf on the roof surface, plus the dead load weight of the deck construction and the test beams. This resulted in a total load of 12,346 lb. for the wood beam, and 12,432 lb. for the steel beam. The difference in total load is due to the lesser weight of the wood beam. The 7 x 21 in. wood beam was selected because it met the requirements of the design. The induced stress was 1552 psi, and the calculated deflection was 2.32 in., or L/224. The 16WF40 steel beam was selected because it met the requirements of the design and is a stock item. The calculated deflection was 1.51 in., or L/344. The induced stress was 12,524 psi.

Test Methods Used: The furnace supplied heat by gas burners that were controlled to conform to the ASTM E119 time/temperature curve.

Report Observations: Temperatures and deflection were measured inside the furnace.

Report Summary: The wood and steel deflection data are summarized in the following table. Time listed is time after burners were lit.

Time (min.)	Temperature Near the Beam (°F)	Steel Beam Deflection (in.)	Wood Beam Deflection (in.)
6	894	2.0	approx. .25
14	1194	8.5	approx. 1.0
20	1279	11.75	approx. 1.5
29	1422	35.5	2.25

Table 13. Wood and Steel Deflection Data.

At 30 min. of exposure, the steel no longer supported the roof panel.

The wood beams supported the full design load throughout the test, with a maximum deflection of 2.25 in. at 30 min. After 30 min. of fire exposure, 75% of the original wood section remained undamaged and the beam continued to support the full design load.

> **Comments:** THIS TEST MAKES A DIRECT COMPARISON BETWEEN THE PERFORMANCE OF TWO STRUCTURAL MEMBERS. THE BEAMS WERE DESIGNED TO CARRY THE FULL DESIGN LOAD OF 30 PSF. THEREFORE, THIS TEST COMPARES EQUIVALENT STRESS PERFORMANCE BETWEEN THESE TWO STRUCTURAL MEMBERS. THIS TESTING INDICATES THAT ONCE STEEL REACHES APPROXIMATELY 1000° F, ITS ABILITY TO RESIST DEFLECTION DECREASES RAPIDLY. IT IS DIFFICULT TO EXTEND A CONCLUSION BEYOND THIS SPECIFIC COMPARISON, HOWEVER, DUE TO THE SPECIALIZED NATURE OF THIS TEST ASSEMBLY.

4-1.13 Report: Fire Performance of Selected Residential Floor Constructions Under Room Burnout Conditions, NBSIR 80-2134

Author: J.B. Fang

Sponsor: United States Department of Housing and Urban Development

Date: December, 1980

Basic Test Description: All the fire resistance tests were performed in a burn room having a 10.7 x 10.7 ft. floor with a 7.4 ft. ceiling height. A doorway opening measuring

30 in. wide and 80 in. tall was situated in the middle of one of the room walls to serve as the single source of room ventilation. The internal walls of the test room were lined with 5/32 in. thick prefinished and printed, three-ply lauan plywood panels. The wall framing consisted of nominal 1 x 3 furring strips spaced 16 in. on center and secured to concrete block walls. The plywood panels were applied with long edges parallel to the wood furring strips. The household furniture used for each test was that commonly found in a recreation room, and included a sofa, upholstered chair, ottoman, end table, bookcase, and coffee table. The fire load density used for this series of fire tests was 4.7 psf of floor area, which was average for recreation rooms in the basements of single-family homes in the Washington, D.C. metropolitan area. In addition to the furnishings, old record files were added in sufficient quantities to reach the required total fire load density. 10 lb. of paper was also placed atop the coffee table, 4 lb. each on the tops of the ottoman and the end table, and the rest—approximately 170 lb.—on the shelves of the bookcase. An Olefin carpet with foam rubber backing was placed on top of a protective layer of 5/8 in. thick Type X gypsum wallboard covering the concrete floor. The total fire load ranged from 7.3 to 7.8 psf, with an average of 7.6 psf of floor area.

For each test, a selected floor-ceiling assembly, 12 x 12 ft., was built over the top of concrete block walls in the burn room, carried uniformly distributed loads, and was subjected to these fire conditions. A portion of the assembly exposed to the room fire below was 10.5 x 10.5 ft. Seven floor-ceiling assemblies were tested. Tests 1 - 4 were unsheathed, and 5 - 7 were protected. The protected tests (5 - 7) are described in **Chapter 4-2: Fire Endurance Performance of Single Membrane Protected Assemblies**.

Test 1: Test 1 was conducted on 2 x 8 wood joists placed parallel to the wall containing the doorway opening, and spaced 16 in. on center. Each joist was kiln-dried, construction grade #2, Eastern Spruce. The joists were cut to 11.7 ft. in length, and secured to 2 x 8 wood rim joists. The rim joists were toe-nailed to nominal 2 x 8 sill plates resting on the concrete blocks. A single layer of 5/8 in. thick plywood subfloor was laid perpendicular to the joists. An olefin carpet with foam rubber backing was fixed to the plywood deck. The load applied was 40 psf, which represented 69% of the maximum allowable stress.

Test 2: Test 2 was conducted on C-shaped, galvanized steel joists, 7.25 in. deep, with a 1.75 in. flange, a 9/16 in. lip, and 18 gauge thickness. The joists were spaced 24 in. on center, beginning with one joist positioned along the centerline of the room width. Each joist was cut 11.7 ft. long and secured to a 2 x 8 wood rim joist. A galvanized steel strap was installed at mid-span, in accordance with the structural design for the steel joists. The rim joists were secured to a 2 x 8 wood sill plate. A 5/8 in. thick plywood floor was attached. An Olefin carpet with foam rubber backing was applied over the plywood. The load applied to this assembly was 72 psf, which represented 100% of the maximum allowable stress.

Test 3: Test 3 was a C-shaped galvanized steel joist, 7.25 in. deep, with a 1.75 in. flange, and an 18 gauge thickness. The joists were spaced 32 in. on center, and were fastened to a 2 x 8 rim joist. The rim joists were secured to a 2 x 8 sill plate. A single layer of 3/4 in. thick tongue-and-groove plywood was installed perpendicular to the joists. A piece of Olefin carpet with foam rubber backing was installed over the plywood deck. A load of 40 psf was applied to the joists, which represented 74% of the maximum allowable stress.

Test 4: Test 4 was constructed in a manner similar to that for Test 1, except that the wood joists were spaced 24 in. on center starting with a floor joist positioned along the horizontal centerline of the room width. A 23/32 in. thick underlayment grade plywood was placed on the joists. Southern Pine #2 bridging was installed along the mid span. The wood joists used were Southern Pine, Construction Grade #2--Medium Grain. A 40 psf load was applied to the joists, which represented 100% of the maximum allowable stress.

Test Methods Used: For each test, the assembly was loaded with 5-1/2 x 6 x 8 in. steel blocks, weighing 50 lbs., to the prescribed load. The uniform load was normally applied a few days prior to the fire test. The ignition source used for all experiments was a section of newspaper weighing 0.9 lb., and placed along the central backrest on seat cushions of the sofa supported by a steel frame holder to ensure reproducible ignition conditions between tests. The paper was conditioned to equilibrium in a room controlled at a dry bulb temperature of 23 $\pm$ 3°C, and a relative humidity of 50 $\pm$ 5% prior to the test. The fire test was started by remotely igniting the newspaper using an electric heating element and a book of paper matches.

Photographic and videotape records and visual observations were made of the progress of the room fires, including the burning characteristics of the assembly, flamethrough and collapse of the structural elements. The fire was allowed to burn until structural failure occurred in the test assembly.

The room air temperatures were monitored at eight locations, including seven within the test room and one in the doorway opening. A total of 25 thermocouples were arranged at various heights in vertical thermocouple trees at the seven locations inside the room.

The surface temperatures of the plywood paneling and the concrete block walls were determined at 16 locations attached to the exposed and unexposed surfaces at selected locations. One location was at the front wall, eight were distributed over the back wall in the vicinity of the ignition source, and three were situated at the left and right plywood paneling walls.

The temperatures on the exposed side of the test assembly were measured using nine thermocouples. Eight thermocouples were installed on both top and bottom flanges of each selected floor joist. One thermocouple was placed on the fire exposed face of the ceiling at the center of the room for test assemblies with a gypsum board ceiling. Finally, on the unexposed surface, three additional thermocouples beneath pads were used to measure temperatures at the points which appeared to be the hottest during the test.

Deflection measurements were made during the test at the mid point and quarter points of each joist. A total of six points for each test structure were measured.

Levels of static pressure which developed within the room were continuously measured. Two locations were used for these pressure measurements: one was 0.7 ft. from the front block wall at 0.2, 0.6, 1.3, 2.7, 3.7, and 5.1 ft. below the ceiling; the other was at the mid-width of the paneling wall at 0.2 ft. from the ceiling.

Total heat fluxes were measured at selected locations using five Gordon foil type, water-cooled heat flux gauges.

Horizontal velocities of the air entering and leaving the fire room through the doorway opening were monitored with six bi-directional flow probes in conjunction with variable reluctance, differential pressure transducers and carrier demodulators. The optical density of smoke was measured at various locations by determining the attenuation of a collimated light beam passing through effluent gas and impinging on a photodetector.

Combustion gas venting from the fire room was sampled at four locations for measuring concentrations of selected gas types.

A total of 136 sensors were automatically read and recorded at a rate of 8 sec. per scan during the entire duration of the test.

Report Summary:

Test 1: Flame penetration occurred near the joint between two sheets of plywood subfloor in the southwest corner, located above the right arm of the sofa, and was observed at 10 min., 17 sec. There was a load failure with steel blocks falling onto the floor, resulting from structural collapse of the centrally located wood joist at 10 min., 43 sec. The average surface temperature of the carpet finish floor increased rapidly to 206°C at 11 min., 7 sec., and the individual temperature readings at two locations exceeded 240°C at 10 min., 59 sec.

Test 2: Failure of the assembly took place at 3 min., 47 sec., by the passage of flames to the unexposed surface near the center of the assembly. The deflection of the test floor measured at the center point showed a rapid increase after 3 min., 31 sec., and the central joist collapsed at the same time as flamethrough. The temperature rise on the unexposed surface in the vicinity of burn through reached 163°C at 3 min., 41 sec.

Test 3: Flame penetration near the west quarter point along the center joist on the west side of the tongue-and-groove joint between two sheets of plywood underlayment located above the right seat cushion of the sofa was observed at 3 min., 58 sec. Based on results of deflection measurements, structural collapse of the center joist occurred at 3 min., 59 sec. One thermocouple positioned on the carpet in the neighborhood of the flamethrough region indicated a steep temperature rise to 239°C at 4 min., 7 sec. The average temperature rise of the surface thermocouples was less than 45°C at the end of the test.

Test 4: Deflection measurements at the center of the assembly showed a rapid increase at 11 min., 52 sec., and the centrally located joist fractured, causing the steel blocks to fall into the fire room at 12 min. Passage of flames and hot gases through the assembly to the unexposed surface occurred at 12 min., 2 sec. in an area near the center of the assembly on the southwest side, somewhat away from thermocouple locations. The average surface temperature on the carpet flooring increased rapidly to 462°C at 12 min., 8 sec., and the individual temperature rise of greater than 196°C occurred almost at the same instant as flame penetration.

Report Conclusions: Based on the experimental results, the following observations can be made regarding the unsheathed assemblies: The unsheathed, light gauge, steel framed assemblies allowed passage of flames, and suffered structural collapse in 4 min., compared to approximately 10 min. for the exposed wood frame floors. Under fire exposure, wood frame floors deflected at a slower rate as compared to steel framed floors; their ultimate collapse is due to the gradual reduction in cross-sectional area of floor joists caused by the charring and burning of wood. Failure due to passage of flames to the unexposed surface of the floor structure resulted from the increased deflection of floor joists with elevated temperatures, which promoted joint separation and developed openings in the plywood subfloor.

The results are summarized in the following tables:

	Structural Elements					Time to		Time to Unexp. Temp. Rise		Maximum Deflection	
Test No.	Floor Joists*	Plyw. Subf. thick. (in)**	Joist Spacing (in.)	Applied Load (psf)	Max. Allow. Stress (%)	Flame-Through (m:s)	Struct. Failure (m:s)	Avg. Temp 139°C (m:s)	1-Point Temp. 181°C (m:s)	Time (m:s)	Center Point (in.)
1	Wood	5/8	16	40	69	10:17	10:43	11:02	10:56	10:43	14.36
2	Steel	5/8	24	72	100	3:47	3:47	3:50	3:41	3:47	14.25
3	Steel	3/4	32	40	74	3:58	3:59	N.R.	4:04	4:07	13.0
4	Wood	23/32	24	40	100	12:02	12:00	12:08	12:02	12:00	6.9

* Wood Joists, nominal 2 x 8 Steel Joists, 1.75 X 7.25 in. X 18 gauge, Super-C. Span of all joists was 10.67 ft.; all assemblies were unsheathed.

** An olefin carpet with foam rubber backing was installed over the plywood subfloor.

N.R. = not reached

Table 14. Test Results for NBSIR 80-2134.

Test No.	Initial Room Temp (°C)	Ambient Relative Humidity (%)	Time to Flame Appearance on Newspaper (min.)	Time from Flame Appearance to: Room Flashover — Ignition of News-paper (min.)	Ignition of Filter Paper (min.)	20 kW/m^2 on Floor (min.)	Flames Emerging from Door-way (min.)	Ignition of Carpet (min.)	Termination of Test (min.)
1	28	50	0.75	1.50	1.53	1.35	1.28	1.58	11.22
2	27	60	0.22	2.30	2.33	2.18	1.82	2.37	4.95
3	27	52	0.15	2.32	2.35	2.37	2.02	2.50	4.38
4	26	54	0.13	2.47	2.52	2.34	1.89	2.52	12.13

Table 15. Continuation of Test Results for NBSIR 80-2134

Comments: The foregoing tests are the only series of tests reviewed that have been conducted which simulate actual fire conditions. The times to failure are very similar to those observed in the ASTM E119 tests. Deflection performance of the assemblies is similar as well. The applied load influences the fire endurance test results. Results of the steel joist tests show greater deflection under heavier applied load. Wood joist performance shows greater deflection under 69% of the design load. The Southern Pine joist system was at 100% of the maximum allowable stress, yet the failure times rose and the deflection decreased, which is contrary to logical expectations. This points out the fact that basing decisions on single tests can lead to results that are not expected. Changing any variable in a test program (e.g., species of lumber, moisture content, etc.) can change the test results for unsheathed assemblies. It further suggests that any comparative unsheathed testing should be performed under identical conditions (i.e., conditioning, stress levels., etc.) in order to gain insight on comparative performance.

4-1.14 Report: Fire Endurance Tests of Selected Residential Floor Constructions, NBSIR 82-2488

Author: J.B. Fang

Sponsor: United States Department of Housing and Urban Development

Date: April, 1982

Basic Test Description: This series of tests was conducted with a pilot furnace which had internal dimensions of 8 x 9.6 ft., and a height of 9.35 ft. The furnace was fired with natural gas. Eight nozzle-mixing gas burners were distributed evenly over the floor area into two rows of four burners each, and mounted to the furnace in the upright position.

Each floor/ceiling assembly measured 8 x 10 ft., and was laid atop the specimen support frame. For Tests 1 and 2, which were protected tests, the applied load was calculated to stress the floor joists to a maximum total deflection and bending moment permitted by the design specifications. This was done in order to compare the fire performance results of the floor assemblies evaluated in the test furnace with those obtained in the room tests. The structural loading for Tests 3 - 5 and 8 - 10 were selected to develop the same magnitude of bending stresses in the floor joist as those produced in Tests 3, 4, and 7 (at 40 psf) in the series. A live load of 54 psf—which corresponds to approximately 93% of the respective maximum load based on the maximum allowable bending stresses for wood joists—was applied to each assembly. Tests 3 - 7 and 9 were all conducted on unsheathed 2 x 8 wood joists. Assembly 10 was a 7.25 in. deep, 18 gauge steel C-joist test. Tests 3 - 5, 9 and 10 used the new time/temperature curve developed from the room tests. Tests 6 and 7 used the ASTM E119 time/temperature curve. For specific test information, see the summary table. Tests 1, 2, and 8 were protected by Type X gypsum wallboard, and will be described in **Chapter 4-2: Fire Endurance Performance of Single Membrane Protected Assemblies**.

Test Methods Used: Each test assembly was built and installed in a test frame of the furnace. Several days prior to fire exposure, the assembly was loaded uniformly with steel blocks to the prescribed load. Unprotected, fast response thermocouples were used to provide the mean gas temperature, which followed the time/temperature curves through manual control of the gas flow to the burners.

Report Observations: Photographic and videotaped records were made of the burning characteristics of each test assembly, including floor deflection, time to burn through, and time to structural failure. Temperatures in the furnace were monitored by using nine commercial metallic sheathed mineral-insulated fast response thermocouples, and nine ASTM E119 standard protected furnace thermocouples. Deflection of the test assembly during the test was measured at the midpoints and selected quarter points of the three centrally located floor joists. Static pressure at various locations inside the test furnace were continuously monitored through four steel pipes extending through the east and west walls of the furnace, with their open ends flush with the wall surfaces. Total heat flux occurring at selected locations were measured with three Gordon-foil-type, water-cooled heat gauges. Continuous gas samples were drawn from the flue gas stream with steel tubing. Outflow gases were analyzed for oxygen, carbon dioxide, and carbon monoxide. The output signals from the thermocouples and various transducers were recorded every eight seconds, and visual observations were recorded.

Report Summary:

Test 3: Penetration of the plywood subfloor/carpet flooring was observed at 6 min., 4 sec. Floor deflection showed a significant increase at 5 min., 10 sec. At test end, the total deflection was 2.24 in. The entire test structure collapsed and fell into the furnace at 6 min., 53 sec. Individual temperature rise on the unexposed surface near the center of the test floor exceeded 181°C at 6 min., 34 sec. Average temperature rise was less than

49°C at the time of test termination. The fire exposure for this test was the new time/temperature curve based on room tests. A high level of excess air was used.

Test 4: Passage of flames and hot gasses through the assembly to the unexposed side occurred at 6 min., 7 sec. Additional penetration of flames to the unexposed surface was observed at 6 min., 40 sec. at several locations near the center of the test floor. Structural collapse of the center joist—based on floor deflection measurements—occurred at 7 min., 52 sec. Maximum deflection was 10.8 in. at 7 min., 52 sec. One thermocouple in the vicinity of the center of the assembly indicated a steep temperature rise to 247°C at 6 min., 56 sec. The average surface temperature rise of the unexposed face exceeded 139°C at 7 min., 13 sec. The fire exposure for this test was the newly developed time/temperature curve. A high level of excess air was used.

Test 5: Flame penetration of the carpeted plywood subfloor occurred at 7 min., 0 sec. The rate of deflection measured at the center of the test floor showed an increase at 8 min., 48 sec. Maximum deflection was 10.7 in. at 10 min., 48 sec. One surface thermocouple located on the carpet surface near the location of burn through registered 223°C at 10 min., 8 sec., and the maximum average temperature rise of the unexposed surface was 177°C at the end of the test. The fire exposure for this test was the new time/temperature curve. A high level of excess air was used.

Test 6: Penetration of flames through the unexposed surface was observed at 16 min., 8 sec. Floor deflection measured at the center of the test assembly increased rapidly at 14 min., 50 sec., and the total deflection was 6.9 in. when the test was terminated at 16 min., 50 sec. A single thermocouple on the carpet floor near the center of the assembly indicated 204°C at 16 min. The average value read by the surface thermocouples on the unexposed side at test termination was 202°C above its initial value. The ASTM E119 time/temperature curve was used. A low level of excess air was used.

Test 7: Flames penetrated the carpeted plywood deck near the center joist at 17 min., 35 sec. Deflection was 11.9 in. at 17 min., 40 sec. The maximum individual thermocouple temperature on the unexposed face was 103°C at 17 min., 10 sec. Average surface temperature rise was 217°C over ambient after 18 min. of test duration. Assembly 7 was tested using the ASTM E119 time/temperature curve. A low level of excess air was used.

Test 9: Failure of the test assembly due to flames passing through the unexposed surface was observed at 9 min., 9 sec. Floor deflection measured at the center of the test assembly increased dramatically at 8 min., 30 sec. The maximum deflection was 3.7 in. at 10 min. The maximum temperature rise measured by one thermocouple on the unexposed surface near the center of the test floor was 181°C at 9 min., 38 sec. Average temperature rise of the unexposed surface was 74°C at the time of test termination. The fire exposure for this test was the new time/temperature curve. A low level of excess air was used.

Test 10: The fire burned through to the unexposed surface, causing system failure at 4 min., 38 sec. Deflection measurements indicated a rapid increase at 2 min., 45 sec.

The maximum deflection was 9.1 in. at 5 min., 50 sec. A single thermocouple near the center of the test structure exceeded 181°C above ambient temperature at 4 min., 24 sec. The average temperature rise of the unexposed surface was 131°C at 5 min., 48 sec. The fire exposure for this test was the new time/temperature curve. The steel joists were stressed to 68% of their maximum allowable stress for this test. A low level of excess air was used.

A summary of these tests follows:

Test No.	Floor Joists*	Joist Spacing (in.)	Applied Load (psf)	Max. Allow. Stress (%)	Fire Expos. **	Level of Excess Air	Time to Flame-Through (m:s)	Time to Struct. Failure (m:s)	Time to Unexp. Temp. Rise: Avg. Temp 139° F (m:s)	Time to Unexp. Temp. Rise: 1-Point Temp. 181° F (m:s)	Maximum Deflection: Time (m:s)	Maximum Deflection: Center Point (in.)
3	Wood	24	54	94	N.D.	High	6:04	6:53	N.R.	6:39	7:00	2.2
4	Wood	24	54	93	ASTM	High	6:07	7:52	7:13	6:53	8:00	10.8
5	Wood	24	54	93	N.D.	High	6:53	7:36	10:39	10:06	10:48	10.7
6	Wood	24	54	93	ASTM	Low	16:08	14:42	16:46	16:00	16:50	6.9
7	Wood	24	54	93	ASTM	Low	17:35	13:10	N.R.	17:08	18:10	12.2
9	Wood	24	54	93	N.D.	Low	9:09	8:48	N.R.	9:38	10:00	3.7
10	Steel	32	55	68	N.D.	Low	4:38	2:48	5:48	4:24	5:50	9.1

* Wood joists: Southern Pine, nominal 2 x 8; Steel Joists: Super-C, 1.75 in. x 7.25 in. x 18 gauge; All assemblies have plywood subfloor thickness of 23/32 in., with no gypsum board ceiling

** N.D. = Newly Developed; ASTM = ASTM E119

Table 16. Test Results for NBSIR 82-2488

Report Observations: The new time/temperature curve—which represents a high intensity, short duration fire exposure—is regarded as a more realistic representation of the severity of room fires found in residential occupancies.

The purpose of this testing was to compare the newly developed time/temperature curve with the ASTM E119 curve. Tests were also run with low and high percentages of excess air in order to determine the effect of oxygen concentration in the furnace on the failure time of combustible construction. The following observations were made of the unexposed assemblies from these tests:

- Wood joist floors exposed to the newly developed fire conditions had a shorter time to failure compared with residential room fire tests of the same floor construction. This was due primarily to increased burning rates of combustible materials in the test structure with excess air present in the test furnace.

- Individual test assemblies tested under the newly developed time/temperature curve resisted flame penetration for approximately 40% less time than those using the ASTM E119 curve.
- Under the new fire exposure condition, unsheathed wood frame floors allowed passage of flames at 9 min., compared with 4.5 min. for the exposed light gauge steel frame floor. The seams of the floor opened due to the sag of the steel joists.
- The fast response thermocouple has a shorter lag time and provides a better indication of the true furnace temperature when compared to the ASTM E119 thermocouple—especially during the early stages of the test.

Comments: This testing was performed to compare a time/temperature curve developed from room fire tests with the ASTM E119 time/temperature curve. The only tests that make an exact comparison are Tests 6, 7, and 9. The average time to structural failure for the ASTM E119 tests was 13 min., 56 sec. Structural failure for the new time/temperature curve under similar test conditions was 8 min., 48 sec. Thus, use of the new curve results in more rapid failure of assemblies. This applies to time of flamethrough of the sheathing as well. Structural failure time due to the new time/temperature curve occurred approximately 40% earlier than when the ASTM E119 curve was used. This test series also investigates the amount of air supplied in a test. The greater the amount of air available to the furnace in testing, the more quickly an assembly will fail. Therefore, when a fire is well ventilated, supplying a greater amount of air to the fire source, an assembly will fail more rapidly.

4-1.15 Report: Fire Endurance Tests of Plywood on Steel Joist Floor Assemblies, With and Without Ceiling, NBSIR 73-14-1

Authors: H. Shoub and B.C. Son

Sponsor: National Bureau of Standards

Date: March, 1973

Basic Test Description: The area and size of the floor assembly was 11 ft. x 9 in. x 17 ft., 11 in., and consisted of 3/4 in. tongue-and-groove underlayment-grade plywood over 6 x 1.75 in. cold-rolled steel "C"-joists, spaced 24 in. on center. Half of the plywood surface—8 ft., 11.5 in.—was covered with 3/8 in. nylon pile carpeting with jute backing, laid over 1/4 in. rubberized hair padding. The other half was not covered. A load equivalent to 51.4 psf was applied to the floor specimen.

Test Methods Used: Testing was generally performed in accordance with the requirements of ASTM E119 for floors and roofs.

Test Results: Failure occurred at 3 min., 15 sec. when flamethrough occurred at the unexposed surface, followed by collapse of the entire assembly at 3 min., 45 sec.

> **Comments:** THE DETAIL AVAILABLE FOR THIS TEST IS LIMITED; THEREFORE, IT IS NOT KNOWN IF THE LOAD APPLIED TO THE FLOOR WAS THE MAXIMUM DESIGN LOAD FOR THE JOIST WAS APPLIED. IF IT WAS, IT CAN THEN BE COMPARED TO THE OTHER ASTM E119 TESTS ABOVE. THIS MAY, THEREFORE, EXTEND THE RANGE FOR CHANNEL-SHAPED STEEL JOISTS DOWN TO 3 MIN. 45 SEC.

4-1.16 Report: Fire Endurance Test of a Steel Sandwich Panel Floor Construction, NBSIR 73-164

Author: B.C. Son

Sponsor: National Bureau of Standards

Date: April, 1973

Basic Test Description: The structural frame of the floor assembly consisted of 6 x 3 in., 14 gauge steel "C"-joists as stringer beams, the joists being 48 in. on center. The overall size of the assembly was 10 ft., 7.25 in. x 17 ft., 11 in. The sandwich panels that were applied to the top of the joists were 3 in. thick, having a paper honeycomb core, with a top surface of 3/8 in. C-D plugged interior grade plywood, and a bottom surface of 26 gauge, galvanized sheet steel. Carpeting was bonded to the plywood. A 40 psf load was applied to the floor assembly during the test.

Test Method Used: The test procedures followed ASTM E119.

Test Results: Failure by flamethrough occurred at a joint between two sandwich panels at 8 min., 45 sec., followed by structural failure at 9 min.

> **Comments:** THIS ASSEMBLY WAS TESTED UNDER A TYPICAL FLOOR LIVE LOAD. THEREFORE, IT CANNOT BE COMPARED TO OTHER TESTING WHERE THE FULL DESIGN LOAD WAS APPLIED.

4-1.17 Report: Fire Testing of Nail Plate-Connected Wood Beams

Authors: B. Roald and E. Aasheim

Sponsor: Norwegian Institute of Wood Technology

Date: 1988

Basic Test Description: The testing consisted of truss plate-joined joists that were combined into beams. Four beams were tested, each 4.6 m long. The beams consisted of:

- Beam 1: four joists, 2.87 in. x 7.79 in. with 1.53 in. laminated veneer lumber (LVL) attached to the bottom.
- Beam 2: four joists, 2.87 in. x 7.79 in. with 1.53 in. LVL attached to the top and bottom.
- Beam 3: two joists, 1.88 in. x 7.75 in. unprotected.
- Beam 4: three joists, 1.82 in. x 7.75 in. with 1.53 in. LVL at the top and bottom.

Each single joist consisted of two parts connected in the center with truss plates. The truss plates were gang nail GN-T150, 6.9 in. x 13.8 in. x 0.6 in. Loads were applied to the four beams to produce a bending moment at the joint location. Due to the load application, beams were exposed to bending tension on the upper side and bending compression on the lower side. The load capacity was not limited by the wood, but by the truss plates.

Test Method Used: The fire test was conducted for 60 min. following the standard time/temperature curve of ISO 834.

Report Observations: Thermocouples were located on each truss plate, and placed directly under the truss plate and at the end of the teeth, so that there were 12 thermocouples on each truss plate.

Report Summary: The results are summarized in the following table:

Beam	Bending Moment (ft.-lbs.)	Failure (min.)
1	7,374	55
2	7,674	*
3	2,957	20
4	4,435	50

* Failure did not occur during the test period of 60 min. After 60 min., the load was increased, and the beam failed at a bending moment of 10,620 ft.-lbs.

Table 17. Test Results for Norwegian Institute of Wood Technology Test.

By placing two or more nail plate-connected beams together, it is possible to achieve improved fire resistance compared to a single nail plate-connected beam.

> **Comments:** THIS REPORT OBSERVES THAT MULTIPLE MEMBERS PERFORM BETTER THAN A SINGLE PLY MEMBERS. THIS MAY BE DUE TO THE FACT THAT THE INTERIOR TRUSS PLATES ARE PROTECTED BY THE WOOD SURROUNDING THEM. THIS STUDY ALSO BUILDS ON THE COMMON EUROPEAN USE OF SACRIFICIAL WOOD TO PROTECT CONNECTIONS. IT IS PROBABLE THIS HAS APPLICATION FOR MULTI-PLY GIRDER TRUSSES (AT LEAST THREE OR MORE CONNECTED TRUSSES) USED BY THE MPC WOOD TRUSS INDUSTRY. IT IS CONCEIVABLE THAT THESE TRUSSES PERFORM BETTER UNDER FIRE CONDITIONS THAN SINGLE TRUSSES SPACED WIDELY (GREATER THAN 4 FT. ON CENTER). HOWEVER, IT IS NOTED THAT ISO 834 INCORPORATES POSITIVE PRESSURE AND UTILIZED BARE THERMOCOUPLES, BOTH OF WHICH MAY EFFECT THE RESULTS OF THIS TEST. CARE MUST BE TAKEN WHEN COMPARING THESE RESULTS WITH RESULTS FROM OTHER TESTS WHICH USE DIFFERENT TEST STANDARDS.

4-1.18 Report: Fireball Tests of Open Webbed Steel Joists

Author: T.E. Waterman, IIT Research Institute

Sponsor: General Services Administration

Date: May 15, 1977

Basic Test Description: Testing resulted from the General Services Administration's concern about storage of records and the potential for fire. The use of high-temperature sprinkler heads permitted areas above the fire to experience temperatures of approximately 1600° F for up to ten minutes. Anticipating that this exposure may cause joist failure, GSA sought to experimentally determine temperatures reached by joists typical of those found in GSA's record centers. Exposing fire temperatures were chosen to approximate the maximum temperatures that were measured by Factory Mutual Research Corporation (FMRC) Test F. Failure was deemed to occur when a joist member temperature exceeded 1100° F. Joists were fabricated for this test to be representative of open webbed steel joists found in the record centers. Each joist was made with a 7 ft. upper chord and a 6 ft. lower chord, and was 12 in. deep.

Test Methods Used: Five instrumented, representative joists were held against the ceiling of a 10 x 15 ft. room. The joists were supported at each end by protected steel frames. Fire in the test room was provided by propane diffusion burners placed near the floors. A recorder measured the temperature at six ceiling locations.

Report Observations: Temperatures on the various components—upper chord, web, and lower chord—of each unsheathed joist were measured. Unsheathed joists generally reached temperatures between 1400 and 1600° F when exposed to the FMRC Test F time/temperature curve. The connections between the chords and the webs remained cooler than the other portions of either chord or web. The flanges of the upper chords in contact with the ceiling remained cooler than the tope chords that were exposed to the fire at the ceiling corrugations.

Report Summary: Based on this test, no joist tested would have met the temperature limitation of 1100° F.

> **Comments:** THIS IS AD-HOC TESTING PERFORMED FOR A SPECIFIC PURPOSE. A DIFFERENT TIME/TEMPERATURE CURVE AND FAILURE CRITERIA IS INTRODUCED. BECAUSE OF THIS, IT IS VERY DIFFICULT TO RELATE THIS TYPE OF TEST TO ANY OF THE OTHER UNSHEATHED TESTS PERFORMED.

4-1.19 Report: BMS 92 Fire Resistance Classifications of Building Constructions

Authors: Subcommittee on Fire Resistance Classifications of the Central Housing Committee on Research, Design, and Construction

Sponsor: United States Department of Commerce and the National Bureau of Standards

Date: October 7, 1942

Basic Test Description: Testing was performed on joists of 2 x 10 Southern Pine or Douglas fir #1 Common or Better Grade, using a subfloor of 3/4 in. wood sheathing, diaphragm of asbestos paper, and finish tongue-and-groove wood flooring. The ratings apply for loadings developing not more than 1000 psi maximum fiber bending stress in the joists. All constructions were rated as combustible because of wood supports and floorboards. Spacing is assumed to be 16 in. on center.

Test Methods Used: The fire tests were conducted in accordance with the standard Specifications for Fire Tests of Building Construction and Materials, American Standards Association (ASA) No. A2-1934. The results of fire tests conducted at the National Bureau of Standards were used as a basis for the ratings. The ratings, in general, were taken directly from test results, and represent the lower average of results.

Report Observations: The ultimate fire resistance time period for exposed wood joists was 15 min.

> **Comments:** THESE TESTS APPEAR TO BE SOME OF THE EARLIEST TESTS PERFORMED ON FLOOR SYSTEMS. THE KEY TO THESE TESTS WAS THE CRITERIA LIMITING STRUCTURAL MEMBERS TO 1000 PSI MAXIMUM FIBER BENDING STRESS. THIS RESTRICTION ON FIBER BENDING STRESS IS PROBABLY THE REASON FOR THE FIRE RESISTANCE OF WOOD JOISTS BEING 15 MIN. A 2 X 10 DOUGLAS FIR OR SOUTHERN PINE #1 OR BETTER GRADE OF LUMBER HAS A FIBER BENDING STRESS EXCEEDING 1000 PSI USING CURRENT DESIGN VALUES.

Test	Structural Member	Spacing	Assemb. Rating (min:sec)	Structural Failure (min:sec)	Loading (psf)	Failure Analysis
IFSI	2 x 10	16 in. o.c.	9:00[1]	> 13:00	31.0	System sagged/gave warning
IFSI	I-joist	24 in. o.c.	4:40[1]	4:40	31.0	No sag/warning
IFSI	MPCT[2]	24 in. o.c.	9:00[1]	15:45	31.0	System sagged/gave warning
IFSI	MPSWT[2]	24 in. o.c.	7:30[1]	N/A	31.0	System sagged/gave warning
IFSI	TJL	24 in. o.c.	6:50[1]	9:45	31.0	No sag/warning
J. Mittendorf	I-joist	32 in. o.c.	N/A	3:20	dead ld	"Early Failure"
J. Mittendorf	TJL	24 in. o.c.	N/A	5:20	dead ld	"Early Failure"
J. Mittendorf	MPCT[2]	16 in. o.c.	N/A	1:20	dead ld	"Early Failure"
J. Mittendorf	MPCT[2]	32 in. o.c.	N/A	> 6:00	dead ld	"Early Failure"

[1] Assembly rating is due to deck burn through.
[2] MPCT = Metal Plate Connected Truss; MPSWT = Metal Plate Steel Web Truss; TJL = Trus Joist L-Series Truss.

Table 18. Non-Standardized Ad-Hoc Unsheathed Assembly Tests.

Structural Member	Structural Failure (min:sec)	Avg. Defl. of Floor (in.)	Loading - % Design Stress
9.5 in. I-joist	~ 5:00	3.1	30% of capacity
10 in. MPCT[1]	~ 5:00	2.7	30% of capacity
10 in. MPSWT[1]	~ 5:00	2.75	30% of capacity
2 x 10	> 10:00	1.1	30/40% of capacity

All tests were proprietary; spacing was single element.

[1] MPCT = Metal Plate Connected Truss; MPSWT = Metal Plate Steel Web Truss.

Table 18a. Standardized Ad-Hoc Unsheathed Assembly Tests.

Test	Structural Member	Spacing	Assemb. Rating (min:sec)	Structural Failure (min:sec)	Avg. Defl. at Floor (in.)	Loading (psf) - % Design Stress	Comments
FM FC 209	2 x 10; 23/32"ply. w/vnl[5]	24 in. o.c.	N/A	13:34	2.83	62.1 (100%)	ASTM E119
FM FC 212	2 x 10 ;23/32"ply. w/cpt[5]	24 in. o.c.	N/A	12:06	3.58	62.4 (100%)	ASTM E119
NBS 421346 (2)	2 x 10; 2-½" ply.	16 in. o.c.	N/A	11:38	2.7	63.7 (100%)	ASTM E119
NBS 421346 (4)	2 x 10; 2-½" ply. w/cpt.[5]	16 in. o.c.	N/A	11:38	3.3	63.7 (100%)	ASTM E119
NBS 421346 (9)	2 x 8; ½ in. ply. w/blk[5]	16 in. o.c.	10:00	13:00	7.0	21.0[1] (40%)	ASTM E119
NBS 421346 (10)	2 x 8; 5/8 in. ply. T&G[5]	16 in. o.c.	9:00	13:00	12.0	21.0[1] (40%)	ASTM E119
FPL	2 x 10	16 in. o.c.	N/A	6:30	4.0	79.2[6] (100%)	ASTM E119
FPL	2 x 10	16 in. o.c.	N/A	13:06	N/A	40.0[1]	ASTM E119
FPL	2 x 10	16 in. o.c.	N/A	17:54	1.7	11.35[1]	ASTM E119
FM FC 250	12 in. MPCT[7]	24 in. o.c.	7:30	10:12	11.5	60.0 (100%)	ASTM E119
NFPA Tech Report 1	4 x 14 Wood Beam	3 ft. 7 in. o.c.	N/A	> 13:00[2]	0.5	30.0[1]	ASTM E119
NFPA Tech Report 1	14 in. Steel bar joist	3 ft. 7 in. o.c.	N/A	13:00[2]	18.0	30.0[1]	ASTM E119
FM FC 208	7¼ in. Steel C-joist	24 in. o.c.	7:24	7:30	7.0	69.8 (100%)	ASTM E119
FM FC 211	7¼ in. Steel C-joist	24 in. o.c.	5:12	5:12	10.0	69.8 (100%)	ASTM E119
NBSIR 73-141	6 x 1¾ in. C-joist	24 in. o.c.	3:15	3:45	N/A	51.4[1]	ASTM E119
NBSIR 73-164	6 x 3 in. 14 ga C-joist	48 in. o.c.	8:45	9:00	N/A	40.0[1]	ASTM E119
NFPA Tech Report 3	7 x 21 Wood Beam	Sngl. Elmt.	N/A	> 30:00[3]	2.25	30.0[1]	ASTM E119
NFPA Tech Report 3	16 WF 40 Steel Beam	Sngl. Elmt.	N/A	30:00[3]	35.5	30.0[1]	ASTM E119
NIWT (1)	11.5 X 9.3 in. Beam	5 PC. Beam	N/A	55:00	N/A	7,374 ft.-lbs.	ISO 834 TPSB[7]
NIWT (2)	11.5 x 10.8 in. Beam	6 PC. Beam	N/A	> 60:00	N/A	7,674 ft.-lbs.	ISO 834 TPSB[7]
NIWT (3)	3.77 x 7.79 in. Beam	2 PC. Beam	N/A	20:00	N/A	2,957 ft.-lbs.	ISO 834 TPSB[7]
NIWT (4)	5.66 x 9.3 in. Beam	3 PC. Beam	N/A	50:00	N/A	4,435 ft.-lbs.	ISO 834 TPSB[7]
BMS 92	2 x 10	16 in. o.c.	15:00	N/A	N/A	N/A	1000 psi mx.F_b ASA A2-1934[7]
IITRI J6397	12 in. Steel Bar Joist	Sngl. Elmt.	N/A	10:06	N/A	dead ld	FMRC Test F;1100°=Fail.[4]

1 Assumed to be a limited load test. Loading not 100% of design load.
2 1/2 in. deflection of wood; 18 in. deflection for steel; 80% of wood undamaged.
3 2.25 in. deflection for wood beam at 30 min.; collapse of steel at 30 min.; 76% of wood undamaged.
4 Time bottom chord reached 100° F is assumed to be failure.
5 vnl = vinyl covering; cpt = carpet covering; blk = 1 x 3 end blocking; T&G = tongue-and-groove.
6 Whether or not this test was at full design load or greater than full design load has been questioned. The structural failure time listed may not be correct.
7 MPCT = Metal Plate Connected Truss; F_b = fiber bending stress; TPSB = Truss Plate Spliced Beam.

Table 19. Standardized Unsheathed Assembly Tests.

Test	Structural Member	Spacing	Assemb. Rating (min:sec)	Structural Failure (min:sec)	Loading (psf) - % Design Stress	Loading (psf)
NBSIR 88-2134 (1)	2 x 8 5/8" ply.	16" o.c.	10:17	10:43	11.3	40 (69%)
NBSIR 88-2134 (2)	7¼"steel C 5/8" ply.	24" o.c.	3:47	3:47	14.25	72 (100%)
NBSIR 88-2134 (3)	7¼"steel C 3/4" ply.	32" o.c.	3:58	3:59	13.00	40 (74%)
NBSIR 88-2134 (4)	2 x 8 23/32" ply.	24" o.c.	12:00	12:00	6.90	40 (100%)

Table 20. Standardized Room Burn Tests.

Test	Structural Member	Assemb. Rating (min:sec)	Structural Failure (min:sec)	Avg. Defl. of Floor (in.)	Comments
NBSIR 88-2488 (3)	2 x 8 23/32" ply.	6:89	6:53	2.24	New T/T curve; high air
NBSIR 88-2488 (4)	2 x 8 23/32" ply.	6:07	7:52	10.8	ASTM E119; high air
NBSIR 88-2488 (5)	2 x 8 23/32" ply.	6:53	7:36	10.7	New T/T curve; high air
NBSIR 88-2488 (6)	2 x 8 23/32" ply.	14:42	14:42	6.9	ASTM E119; low air
NBSIR 88-2488 (7)	2 x 8 23/32" ply.	13:10	13:10	12.2	ASTM E119; low air
NBSIR 88-2488 (9)	2 x 8 23/32" ply.	8:48	8:48	3.7	New T/T curve; low air
NBSIR 88-2488 (10)	7¼"steel C 23/32"ply.	2:48	2:48	9.1	New T/T curve; low air

All assemblies were loaded to 54 psf, which was 93% of their capacity; all spacings were 24 in. on center.

Table 21. New Time/Temperature Curve Evaluation Tests.

Test	Structural Member	Spacing	Assemb. Rating (min:sec)	Structural Failure (min:sec)	Avg. Defl. at Floor (in.)	Loading (psf) - % Design Stress
FM FC 209	2 x 10; 23/32"ply. w/vnl	24 in. o.c.	N/A	13:34	2.83	62.1 (100%)
FM FC 212	2 x 10 ;23/32"ply. w/CPT	24 in. o.c.	N/A	12:06	3.58	62.4 (100%)
NBS 421346 (2)	2 x 10	16 in. o.c.	N/A	11:38	2.7	63.7 (100%)
NBS 421346 (4)	2 x 10; 2-½" ply.	16 in. o.c.	N/A	11:38	3.3	63.7 (100%)
FPL	2 x 10	16 in. o.c.	N/A	6:30	4.0	79.2[2] (100%)
FM FC 250	12 in. MPCT[1]	24 in. o.c.	7:30	10:12	11.5	60.0 (100%)
FM FC 208	7¼ in. Steel C-joist	24 in. o.c.	7:24	7:30	7.0	69.8 (100%)
FM FC 211	7¼ in. Steel C-joist	24 in. o.c.	5:12	5:12	10.0	69.8 (100%)

[1] MPCT = Metal Plate Connected Truss; MPSWT = Metal Plate Steel Web Truss; TJL = Truss Joist L-Series Truss; TPSB = Truss Plate Spliced Beam; F_b = fiber bending stress.

[2] Whether or not this test was at full design load or greater than full design load has been questioned. The structural failure time listed may not be correct.

Table 22. ASTM E119 Unsheathed Assembly Tests at Full Design Load.

4-1.20 Evaluation of Unsheathed Assemblies Testing Performance

Test data available allowing direct comparison between assemblies are represented by eight tests (see Table 22 above). These tests indicate that in unsheathed assemblies, wood joists have greater fire endurance than steel C-joists. The data also indicate that metal plate connected (MPC) trusses have fire endurance times that fall within the range of performance for 2 x 10 joists, if the FPL failure time of 6 min., 30 sec. is accurate. If this test is removed because of unreliable data, then MPC trusses have fire endurance times that fall just below the range of performance for 2 x 10 joists. The MPC truss assembly tested, however, did not have a splice plate located in the bottom chord of the truss. It is expected that this may reduce the time to failure, although by an unknown amount.

Information on testing of unsheathed assemblies suggests that wood charring protects the wood member and aids in the structural performance under fire conditions. This is in contrast to steel, where once the steel member reaches a temperature greater than 1000° F, its strength rapidly decreases.

The National Bureau of Standards (Now called the National Institute of Standards and Technology, or NIST) report NBSIR 82-2488 provided data comparing ASTM E119 to a typical burning room, and a new time/temperature curve based on room burn tests. This report indicates that ASTM E119 overstates fire endurance performance as compared to room burn time/temperature curve performance. However, this should be contrasted with full-scale room fire tests showing 2 x 8 joist fire endurance to be 10 to 12 min., in the range of what would be expected for 2 x 8's exposed to ASTM E119 testing. This indicates that the room burn time/temperature curve may not have been calibrated to replicate room burn performance from which it was derived. These NBS tests can provide baseline data, should there be a desire to develop fire test standards that more accurately replicate actual field conditions. To claim that one time/temperature curve is better than another based on this testing would be premature.

Non-standard test data on unsheathed assemblies provided in this chapter cannot be used to compare performance due to the lack of standardized testing procedures.

ASTM E119 can, however, be used to make comparisons between assembly types when maximum design loads are applied to the test assembly. Currently, model building codes require ASTM E119 tests to be performed on protected assemblies only. Therefore, unsheathed tests that have been performed have no utility from a model code perspective, but do provide additional data for study.

Based on the literature review, there are currently no fire endurance performance criteria for unsheathed assemblies. The Federal Housing Administration (FHA) previously had a 10 min. requirement. It was abandoned in November of 1984 as a result of Office of Management and Budget Circular A119, which stated that all federal agencies must use prevailing voluntary codes and standards where they exist. Therefore, the local or model code requirements would determine unsheathed assembly application and required fire endurance, if any. It is likely this lack of performance criteria is the reason for the small amount of standardized data available on unsheathed assemblies under fire conditions.

Chapter 4-2: Fire Endurance Performance of Single Membrane Protected Assemblies

The following protected assembly tests provide fire performance data on the use of a single layer of gypsum wallboard attached to horizontal structural elements. The most important of these are single layer systems with wallboard attached directly to structural elements. These tests generally result in assembly fire endurance performance between 45 and 60 min.

4-2.1 Report: Underwriters Laboratory Design Number L506

Author: Underwriters Laboratory

Sponsor: Gypsum Association

Date: 1950

Basic Test Description: The test assembly used 1/2 in. thick sheets of fire rated wallboard. This wallboard was applied directly to 2 x 10 wood joists which were spaced 16 in. on center and firestopped. The subfloor applied to the joists was 1 x 6 tongue-and-groove, fastened diagonally or 1/2 in. plywood. The finish flooring was 1 x 4 tongue-and-groove boards or 5/8 in. plywood.

Test Method Used: The test followed ASTM E119 procedures.

Report Observations: The only data provided are in the UL directory. This assembly provides a 3/4-hr. unrestrained rating with finish ratings that range from 15 min. to 20 min., depending on the type of gypsum. The gypsum finish ratings are summarized in the following table:

Company	Finish Rating Time (min.)
Canadian Gypsum Company, Ltd. Type SCX, SHX, WRX	15
Celotex Corporation Type A	18
Celotex Corporation Type B	20
Celotex Corporation Type C	15
Domtar Gypsum Type C	20
Gold Bond Building Products Type FSW-1 or FSW-G	20
James Hardy Gypsum Type 3	20
Republic Gypsum Company, Type RG-1	15
Republic Gypsum Company, Type RG-3	20
United States Gypsum Company Types SCX, SHX, WRX	15

Table 23. Gypsum Finish Rating Times.

> **Comments:** Unfortunately, more detailed data on these tests are not available. If time to failure for these assemblies were known, better comparisons could be made between other assembly types listed in this chapter. These data can be used to compare finish ratings with other assemblies, and provide some estimate of the fire endurance performance of the structural framing after the finish rating temperature has occurred.

4-2.2 Report: Underwriters Laboratories Design Number L520

Authors: Underwriters Laboratories, Inc.

Sponsor: Perlite Institute

Date: August 1968.

Basic Test Description: Fire rated, 5/8 in. thick gypsum produced by Canadian Gypsum Company, Ltd. (Type C); Celotex Corporation; Domtar Gypsum (Type 5); Georgia Pacific Corporation, Gypsum Division (Type GPFS-C); Pabco Gypsum Company (Type C or PG-C); and United States Gypsum Company (Type C or IP-X2) was applied to resilient channels 1/2 in. deep, and spaced 24 in. on center. The resilient channels were attached perpendicular to the 2 x 10 wood joists, which were spaced 16 in. on center and firestopped. A 5/8 in. thick plywood subfloor was attached to the wood joists with a 1-5/8 in. thick Perlite sand concrete finished floor over the subfloor. Glass fiber bat insulation, 3 in. thick, was applied directly over the top of the furring channels.

Test Method Used: ASTM E119.

Report Observations: The only data provided in the UL directory were the unrestrained assembly rating of 45 min. and a finish rating of 21 min.

> **Comments:** These data are included to provide an indication of the effects of adding resilient channels and glass fiber insulation, and using a Type C versus Type X Gypsum board.

4-2.3 Report: Building Research Laboratory 5036

Authors: R.W. Bletzacker, J.G. Birle, and D.A. Lucht

Sponsor: Trus Joist Corporation

Date: July, 1971

Basic Test Description: The test consisted of 9-13/16 in. deep I-joists with 3/8 in. plywood webs and 1-7/16 x 2-9/16 in. flanges having a length of 13 ft., 9 in. The I-joists were spaced 24 in. on center. The floor consisted of a base layer of 3/4 in. Douglas fir plywood and a top layer of 3/8 in. Douglas fir exterior grade plywood. The ceiling consisted of a layer of 5/8 in. USG, Firecode C gypsum wallboard. The wallboard was attached by nails. The bridging was nominal 1 x 3 Southern Pine. Holes were drilled in each of the joist webs in accordance with a 1969 Trus Joist I-series publication.

The completed assembly was allowed to air dry in the normal atmosphere of the laboratory for a minimum of seven days to assure dryness of the joint compound.

A superimposed load of 1,389.6 lbs./joist was applied to the assembly at the start of the test. This load, in addition to the dead load of 192.4 lbs./joist, applied a design allowable shear of 791 lbs. to each joist, based upon data published by the sponsor.

Test Method Used: ASTM E119.

Report Observations: Exposed and unexposed surface observations surface temperatures, furnace temperatures, and deflection performance were all measured.

Report Summary: The test assembly failed to support the superimposed load at 48 min. Average deflection along the centerline of the assembly was 3.6 in. At the termination of the test, the center-most point of the assembly showed a deflection of 4.58 in. No unusual exposed or unexposed surface observations were made.

> **Comments:** This test addresses the fire endurance performance of I-joists. The test duration allows for a 45-minute rated assembly. A calculation of the finish rating has been made from the test data, and is included in the table at the end of this section. This also shows performance somewhat similar to wood joists protected with a single layer of gypsum board, which also achieved a 45-minute rating. The difference is the type of wallboard used—5/8-in. Type C versus 1/2-in. Type X, which is significant. Unfortunately, failure times are not provided for the 1/2-in. type X data; therefore, it is difficult to determine just how significant this performance difference is.

4-2.4 Report: Floor/Ceiling Wood Truss Assembly Design FC-235

Author: W.R. Price and W.F. Shield, Factory Mutual Research

Sponsor: Truss Plate Institute

Date: August 6, 1976

Basic Test Description: The floor assembly consisted of floor trusses, 12 in. deep with nominal 2 x 4 wood chords and webs. The floor trusses were 17 ft., 5 in. long, and were spaced 24 in. on center. The floor was a single layer of 3/4 in. thick plywood with vinyl asbestos tile attached to it. The ceiling was a single layer of 5/8 in. Type FSW (or Type

C) gypsum wallboard, produced by National Gypsum Company, and was secured directly to the bottom chords of the trusses.

Test Method Used: ASTM E119.

Report Observations: Observations were made of the exposed and unexposed surface, deflection measurements of the floor were made, and the temperature of the unexposed surface, plenum and furnace were measured.

Report Summary: The assembly was subjected to a uniformly distributed live load of 50.1 psf, which resulted in a combined live and dead load of 57.4 psf. The deflection at the center of the assembly at 50 min. was 3.5 in. There were no unusual occurrences based on the observations made for both the unexposed and exposed surfaces during the test. The test was terminated at 50 min. when the assembly failed to support the superimposed load. The finish rating was calculated to be 24 min.

> **Comments:** THIS TEST GIVES AN INDICATION OF THE PERFORMANCE OF METAL PLATE CONNECTED WOOD TRUSSES WITH SINGLE LAYER GYPSUM PROTECTION. THE GYPSUM BOARD USED WAS IDENTICAL TO THAT USED IN THE I-JOIST TEST ABOVE.

4-2.5 Report: Floor/Ceiling Truss Assembly Design FC-240

Authors: W.R. Price and W.F. Shield, Factory Mutual Research

Sponsor: Truss Plate Institute

Date: April 13, 1977

Basic Test Description: The floor assembly consisted of floor trusses 12 in. deep with nominal 2 x 4 wood chords and webs, and were 17 ft., 5 in. long. Trusses were spaced 24 in. on center. The floor was a single layer of 3/4 in. thick tongue-and-groove plywood. The ceiling was a single layer of 5/8 in. thick Firecode C gypsum wallboard manufactured by USG secured to furring channels attached to the bottom chords of the trusses. The furring channels, manufactured by USG and designated as RC-1 resilient channels, were installed perpendicular to the trusses, and located 16 in. on center.

Test Method Used: ASTM E119.

Report Observations: Observations of the exposed and unexposed surfaces were made, and the deflection of the floor, the temperature of the furnace, plenum, and unexposed surface, and time of failure of the assembly were recorded.

Report Summary: The assembly was subjected to a uniformly distributed live load of 50.7 psf, which resulted in a combined live and dead load of 57.8 psf. There were no unusual observations of either the exposed or unexposed surfaces during the test. The maximum deflection of the floor occurred at 58 min., where the center-most deflection was 2.13 in. The test was terminated at 58 min., when the assembly failed to support the superimposed load. The finish rating was calculated to be 26 min.

> **Comments:** THIS TEST HIGHLIGHT THE EFFECTS OF RESILIENT CHANNELS ON TRUSSES WITH ALL OTHER FACTORS BEING EQUAL. IN THIS CASE, THE RESILIENT CHANNEL ADDED APPROXIMATELY 6 MIN. TO THE PERFORMANCE OF THE ASSEMBLY, CALCULATED AS FOLLOWS:
>
> $$|(50 - 24) - (58 - 26)| = 6 \text{ MIN.}$$
>
> THIS CALCULATION ACCOUNTS ONLY FOR PERFORMANCE AFTER THE FINISH RATING WAS MET FOR THE MEMBRANE, AND IS CERTAINLY NOT ABSOLUTE FOR ALL CASES. IT ONLY PROVIDES AN INDICATION.

4-2.6 Report: Standard ASTM Fire Endurance Truss Test Project 4816

Authors: R.W. Bletzacker and J.G. Birle, Ohio State University

Sponsor: Trus Joist Corporation

Date: September, 1969

Basic Test Description: The joists tested were 14 in. deep, 15 ft., 10.25 in. long TJL-series joists. The joists were spaced 24 in. on center. The floor consisted of a base layer of 3/4 in. thick Douglas fir plywood and a top layer of 3/8 in. thick Douglas fir plywood. The ceiling consisted of a layer of nail-attached 5/8 in. thick USG sheetrock, Firecode C, gypsum wallboard. Bridging consisting of a 2 x 6 was placed perpendicular to the trusses.

The completed assembly was allowed to air dry in the normal atmosphere of the laboratory for a minimum of seven days to assure dryness of the joint compound. A superimposed design load of 199.8 lbs./lineal ft. was applied at the start of the test. The load was calculated to impose the maximum allowable working stress on the joist.

Test Method Used: ASTM E119.

Report Observations: Observations of both the unexposed and exposed surfaces of the test assembly were made; assembly deflection measurements, temperature of the furnace, plenum and unexposed surface were recorded during the test.

Report Summary: There were no unusual observations noted for the exposed or unexposed surfaces during the test. The center-most deflection at 45 min. was 1.03 in. The average deflection along the centerline of the test at 45 min. was .87 in. The assembly could no longer support the applied load at 48 min. The finish rating for this assembly was calculated to be 22 min.

Comments: THIS TEST PROVIDES ADDITIONAL INFORMATION ON SINGLE LAYER GYPSUM PERFORMANCE ON ANOTHER ENGINEERED SYSTEM. IT APPEARS THAT SINGLE-LAYER 5/8 IN. TYPE X OR TYPE C SYSTEMS DIRECTLY ATTACHED TO THE STRUCTURAL MEMBER GENERALLY YIELD 45-MINUTE FIRE ENDURANCE RATINGS.

4-2.7 Report: Fire Endurance of Light-Framed Miscellaneous Assemblies, Taken from, "Investigation on Building Fires, Part V," By N. Davey and L.A. Ashton

Author: M. Galbreath

Sponsor: National Research Council of Canada

Date: June, 1966

Basic Test Description: The test consisted of regular 1/2 in. Gypsum board directly applied to 2 x 9[1] solid-sawn joists spaced 16 in. on center. One inch nominal tongue-and-groove boards were applied to the top of the joists. The joists were 12 ft. clear span, simply supported. The load applied to the test floor was 60 lbs./ft^2.

Test Method Used: It is assumed that the test method used was ASTM E119.

Report Observations: Thermocouples were used to measure the unexposed surface temperature between floor and the ceiling and the furnace temperature. Test duration, mode of failure, and behavior of the floor were all measured.

Report Summary: The test lasted 33 min., until there was an appearance of flame on the surface, and the assembly collapsed into the furnace. Gypsum board began to fall away from the joists at 18 min., and had completely fallen off the joist at 27 min.

COMMENTS: THIS TEST INCLUDED A VARIETY OF ASSEMBLIES. THIS TEST IS INCLUDED BECAUSE IT GIVES AN INDICATION OF THE CONTRIBUTION TO FIRE ENDURANCE OF 1/2 IN. REGULAR GYPSUM WALLBOARD. THE JOISTS APPEAR TO HAVE BEEN NOMINAL 2 X 10, BUT THEY MAY HAVE ACTUALLY HAD THE FULL 2 IN. DIMENSION RATHER THAN THE 1.5 IN. DIMENSION USED TODAY. ONE PROBLEM WITH USING OLDER DATA IS THAT THE STRUCTURAL WOOD JOISTS USED TODAY ARE SIZED DIFFERENTLY THAN THOSE USED IN 1966. THE JOISTS USED TODAY ARE ALSO DIFFERENT THAN THOSE USED 10 YEARS AGO, DUE TO

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[1] Nominal 2 x 10 is assumed to be meant here.

CHANGES IN TIMBER RESOURCES AND CHANGES IN DESIGN VALUES OVER THAT PERIOD OF TIME.

4-2.8 Report: BMS 92 Fire Resistance Classifications of Building Constructions

Authors: Subcommittee on Fire Resistance Classifications of the Central Housing Committee on Research, Design and Construction

Sponsor: United Stated Department of Commerce and the National Bureau of Standards

Date: October 7, 1942

Basic Test Description: Testing was performed on joists of 2 x 10 Southern Pine or Douglas fir #1 Common or Better Grade, using a subfloor of 3/4 in. wood sheathing, a diaphragm of asbestos paper, and finish tongue-and-groove wood flooring. The ratings apply for loadings developing not more than 1000 lbs./in^2 maximum fiber bending stress in the joists. Spacing is assumed to be 16 in. on center.

Test Methods Used: The fire tests were conducted in accordance with the Standard Specifications for Fire Tests of Building Construction and Materials, ASA No. A2-1934 (precursor to ASTM E119). The results of fire tests conducted at the National Bureau of Standards were used as a basis for the ratings. The ratings, in general, were taken directly from test results, and represent the lower average of results.

Report Observations: 1/2 in. thick gypsum wallboard secured with 1.75 in. No. 12 gauge nails spaced 6 in. on center was found to have a fire resistance rating of 25 min. and provide protection for the wood joists of 15 min. Two layers of 3/8 in. gypsum wallboard using 1.5 in. No. 15 gauge nails spaced 6 in. on center were found to have a fire resistance of 30 min. and provide protection for the wood joists of 20 min.

Comments: THESE TESTS ADDED TO PROVIDE INFORMATION ON REGULAR GYPSUM WALLBOARD PERFORMANCE IN CONTRAST WITH TYPE X OR TYPE C THAT IS TYPICALLY APPLIED.

4-2.9 Report: Fire Performance of Selected Residential Floor Constructions Under Room Burnout Conditions, NBSIR 80-2134

Author: J.B. Fang

Sponsor: United States Department of Housing and Urban Development

Date: December, 1980

Basic Test Description: For general information, see **Section 4-1.13** in **Chapter 4-1: Fire Endurance Performance of Unsheathed Assemblies**.

Test 5: This assembly consisted of C-shaped 18 gauge galvanized steel joists, 7.25 in. deep spaced 24 in. on center. A 23/32 in. subfloor was applied to the top of the joist upon which was placed an Olefin carpet with foam rubber backing. The joists were protected with a 1/2 in. thick regular gypsum board ceiling.

Test 6: This assembly consisted of 12 in. deep x 3.5 in. wide x 11 ft., 8 in. long wood trusses with 2 x 4 wood chords and webs. Flat metal connector plates, fabricated from 20 gauge galvanized steel, were used to secure the webs to the chords. A single layer of 23/32 in. underlayment grade plywood was applied to the top of the trusses. An Olefin carpet with foam rubber backing was installed on top of the plywood. The ceiling was 1/2 in. thick regular gypsum wallboard.

Test 7: The floor framing consisted of nominal 2 x 8 kiln-dried Number 2 Southern Pine joists, spaced 24 in. on center. The joists were 11 ft., 8.5 in. long. The subfloor was 23/32 in. underlayment grade plywood. An Olefin carpet was secured to the plywood deck. The ceiling was 5/8 in., Type X gypsum wallboard.

Report Summary:

Test 5: Failure of the gypsum board ceiling occurred at 13 min., 9 sec. Penetration of flames through the test assembly to the unexposed surface occurred at 15 min., 58 sec. The maximum deflection was 12.8 in. at 16 min., 14 sec.

Test 6: The protective layer of gypsum board utilized as a ceiling finish for the assembly failed at 11 min., 51 sec. Flame penetration occurred at 17 min., 53 sec. near the center of the assembly. The temperature rise of one surface thermocouple positioned on the carpet exceeded 181° C at 17 min., 46 sec. The centrally located joist did not fail until 18 min., 34 sec. due to wood bridging fastened at mid-span. The maximum center point deflection was 12.8 in. at 18 min., 34 sec.

Test 7: The gypsum board ceiling began to fall away in random segments at 23 min., 6 sec. Passage of flames through the assembly to the unexposed surface was recorded at 35 min., 8 sec. Maximum floor deflection was 6 in. at 35 min., 26 sec. The maximum values of average and individual temperature rise on the unexposed carpet surface at the time of test termination were 150° and 108° C, respectively, since thermocouples were away from the burn-through region. Results are summarized in the following tables:

Test No.	Initial Room Temp (°C)	Ambient Relative Humidity (%)	Time to Flame Appearance on Newspaper (min)	Time from Flame Appearance to					
				Room Flashover			Flames Emerging from Door-way (min)	Ignition of Carpet (min)	Termin-ation of Test (min)
				Ignition of		20 kW/m^2 on Floor (min)			
				News-paper (min)	Filter Paper (min)				
5	25	43	0.17	1.73	1.75	1.43	1.51	2.00	16.23
6	24	30	0.10	1.60	1.62	1.42	1.50	1.75	18.80
7	22	42	0.17	1.68	1.70	1.59	1.60	1.97	35.43

Table 24. Test Results for NBSIR 80-2134.

Test No.	Structural Elements: Floor Joists*	Structural Elements: Gypsum Bd. Ceiling (in)**	Applied Load (psf)	Time to: Flame-Through (m:s)	Time to: Struct. Failure (m:s)	Time to Unexposed Temperature Rise: Avg. Temp 139° C (m:s)	Time to Unexposed Temperature Rise: 1-Pt. Temp. 181° C (m:s)	Maximum Deflection: Time (m:s)	Maximum Deflection: Center Point (in.)
5	Steel	1/2	67	15:58	15:58*	15:57	15:55	16:14	12.9
6	Trusses	1/2	67	17:53	18:34	N.R.	17:43	18:34	12.8
7	Wood	5/8 (Type X)	40	35:08	35:18	N.R.	N.R.	35:26	6.0

* Wood Joists, nominal 2 x 8; Steel Joists, 1.75 x 7.25 in. x 18 gauge, Super-C; Wood Trusses, 3.5 x 12 in. prefabricated with 2 x 4 wood chords and webs; Span of all joists was 10.67 ft.; Thickness of plywood subfloors were 23/32 in. An olefin carpet with foam rubber backing was installed over the plywood subfloor. Joists were spaced at 24 in. on center, and loaded to 100% of maximum allowable stress.

** Gypsum board was painted

Table 25. Continuation of Test Results for Test Results for NBSIR 80-2134.

Report Summary: Under fire exposure, wood frame floors deflected at a slower rate as compared to steel frame floors. Ultimate collapse of wood frame floors is due to gradual reduction of the cross-section area of floor joists, caused by charring and burning of wood.

The use of a 1/2 in. thick regular or 5/8 in. thick Type X gypsum board ceiling increased the fire endurance time by 12 and 23 min., respectively, when compared to unsheathed steel and wood joists. No comparison can be made to floor trusses.

> **Comments:** BECAUSE 1/2 IN. REGULAR GYPSUM WALLBOARD WAS NOT UNIFORMLY USED FOR ALL JOIST ASSEMBLIES, IT IS DIFFICULT TO MAKE CLEAR COMPARISONS BETWEEN STEEL AND WOOD JOISTS. IT CAN BE OBSERVED THAT WOOD TRUSSES LASTED APPROXIMATELY 2 MIN. LONGER THAN STEEL JOISTS WHEN PROTECTED BY 1/2 IN. GYPSUM. SINCE THESES WERE ROOM BURN TESTS, THIS ALSO GIVES AN INDICATION OF THE PERFORMANCE OF A PROTECTED ASSEMBLY INSIDE AN ACTUAL FIRE. THE MEMBRANE PROTECTION TIME FOR 1/2 IN. REGULAR WALLBOARD APPEARS TO BE BETWEEN 15 AND 20 MIN. ON STEEL AND WOOD TRUSSES. 5/8 IN. TYPE X GYPSUM WALLBOARD APPEARS TO PROTECT JOISTS FOR LONGER THAN 30 MIN. IN EACH CASE, HOWEVER, THERE WAS SIGNIFICANT DEFLECTION TOWARD THE END OF THE TEST, PROVIDING SOME WARNING OF STRUCTURAL COLLAPSE.

4-2.10 Report: Flame Endurance Tests of Selected Residential Floor Constructions, NBSIR 82-2488

Author: J.B. Fang

Sponsor: United Stated Department of Housing and Urban Development

Date: April, 1982

Basic Test Description: Except as noted, see **Section 4-1.14** in **Chapter 4-1: Fire Endurance Performance of Unsheathed Assemblies**.

For assemblies 1, 2 and 8, the floor framing consisted of 2 x 8 wood joists spaced 24 in. on center. Each joist was 9.17 ft. long. A single layer of 23/32 in. underlayment grade Douglas fir plywood was attached to the top of the joists. An olefin carpet with foam rubber backing was fastened to the plywood deck. The ceiling was a layer of 5/8 in. thick Type X gypsum wallboard. The wallboard was attached to the joists with nails.

Assemblies 1 and 8 used the new time/temperature curve, while Assembly 2 used ASTM E119.

Report Summary:

Test 1: Failure of the gypsum board ceiling was observed at 16 min., 25 sec. Penetration of flames through the unexposed surface occurred at 20 min., 6 sec. A maximum deflection of 1.85 in. was recorded at 20 min., 48 sec. The maximum temperature rise for the average and individual thermocouples attained on the unexposed surface during the test were 56 and 106° C, respectively.

Test 2: The protective layer of gypsum board began to fall at 30 min., 20 sec. Failure of the floor assembly was observed at 34 min. due to passage of flames through the unexposed surface. At 35 min., 4 sec. the floor reached maximum deflection of 13 in. prior to the collapse of the center joist. At 35 min., 8 sec., the average temperature rise was 241° C, and the individual temperature rise exceeded 181° C.

Test 8: The gypsum board ceiling began to fall at 15 min., 40 sec. Penetration of flames through the unexposed surface occurred at 24 min., 22 sec. The maximum deflection of 5.9 in. occurred at 26 min., 30 sec. The individual temperature rise on the unexposed surface exceeded 358° F at 25 min., 3 sec.

The results of these tests are summarized in the following table:

Test No.	Max. Allow. Stress (%)	Fire Exposure	Level of Excess Air	Time to Flame-Through (m:s)	Time to Struct. Failure (m:s)	Time to Unexp. Temp. Rise Avg. Temp 139° C (m:s)	Time to Unexp. Temp. Rise 1-Point Temp. 181° C (m:s)	Maximum Deflection Time (m:s)	Maximum Deflection Center Point (in.)	Avg. Oxygen Conc.* (%)
1[1]	100	New	High	20:06	N.R.	N.R.	N.R.	20:48	1.85	9.8
2[1]	100	ASTM E119	High	34:00	35:20	35:08	34:50	35:28	13.0	12.8
8[1]	93	New	Low	24:22	24:59	N.R.	25:23	26:30	5.9	6.4
Room[1]	—	—	—	—	35:08	—	—	—	—	0.7

[1] All assemblies were 2 x 8 wood joists with plywood subfloors of 23/32 in., gypsum board ceiling 5/8 in. Type X, joists spacing 24 in. on center, and an applied load of 54 psf.; N.R. = Not reached

* For assemblies, measured in the flue gas stream; for room, measured at the top of the doorway.

Table 26. Test Results for NBSIR 82-2488

Report Summary: A protective layer of 5/8 in. thick, Type X gypsum board increased the time to failure by approximately 15 min. for the high intensity, short duration fire exposure; and by approximately 18 min. for the standard ASTM E119 fire exposure when compared to the unsheathed joists under the same conditions.

In the case of the protected assemblies, a steep rise in furnace temperature was observed immediately after the combustible floor was involved in the fire.

Comments: These tests were a follow-up to room fire tests of **Section 4-2.9**. The testing showed that the new time/temperature curve with high levels of excess air caused the protected wood joist assembly to fail more quickly than a similar assembly tested using the ASTM E119 time/temperature curve. This is apparently due to the extremely high temperatures achieved very early in the test. When the new time/temperature curve was used with low levels of excess air, the assembly only increased in endurance time by 4 min. over the test with high excess air level. It is interesting to note that the room test (not the new time/temperature curve) caused structural failure at 35 min., 8 sec., and the ASTM E119 caused structural failure at 35 min., 20 sec. It would appear from this data that the new time/temperature curve is more severe than the room fire upon which the time/temperature curve was developed, given that the difference in performance of the new curve and the room burn was 9 min., 9 sec. ASTM seems to accurately predict performance of the assembly under room burn conditions. Care must be taken when drawing conclusions from these data, particularly since test conditions varied between tests. The only meaningful comparisons that can be made are those which use tests performed under identical conditions. But even then, the testing performed has limited statistical significance.

Test Number	Structural Member[1]	Space O.C. (in.)	Gypsum Type	Ceiling Application	System Used Insulation	Applied Load (psf)[1]	Maximum Deflection (in.)	Finish Rating (min)	Assembly Rating (min)[2]	Standard Test Procedure
UL 506	2 x 10	16	1/2" X	Direct	No	FD	N/A	15	45	ASTM E119
UL 506	2 x 10	16	1/2" X	Direct	No	FD	N/A	20	45	ASTM E119
UL 520	2 x 10	16	5/8" C	R/C Chan.	Yes	FD	N/A	21	45	ASTM E119
BRL 5036	10" I	24	5/8" C	Direct	No	57.5 FD	4.58	23.5	45 (48)	ASTM E119
FM FC-235	12" MPCT	24	5/8" C	Direct	No	57.4 FD	3.5	24	45 (50)	ASTM E119
FM FC-249	12" MPCT	24	5/8" C	R/C Chan.	No	57.8 FD	2.13	26	45 (58)	ASTM E119
BRL 4816	14" TJL	24	5/8" C	Direct	No	99.9 FD	1.03	22	45 (52)	ASTM E119
PFS 88-03[3]	15" MPCT	24	5/8" C	Direct	Yes	FD	N/A	23	45 (52)	ASTM E119
Galbreath[3]	2 x 9	16	1/2" reg	Direct	No	60	N/A	N/A	33	ASTM E119
BMS 92[3]	2 x 10	16	1/2" reg	Direct	No	1000 max	N/A	N/A	25	ASA #A2-1934
BMS 92[3]	2 x 10	16	2-3/8" reg	Direct	No	1000 max	N/A	N/A	30	ASA #A2-1934
NBSIR 80-2131 (5)	7.5"steel C	24	1/2" reg	Direct	No	67 FD	12.9	N/A	16	Room Fire
NBSIR 80-2131 (6)	12" truss	24	1/2" reg	Direct	No	67 FD	12.8	N/A	18	Room Fire
NBSIR 80-2131 (7)	2 x 8	24	5/8" X	Direct	No	40 FD	6.0	N/A	35	Room Fire
NBSIR 80-2488 (1)	2 x 8	24	5/8" X	Direct	No	54 FD	1.85	N/A	20	New T/T
NBSIR 80-2488 (2)	2 x 8	24	5/8" X	Direct	No	54 FD	13.0	N/A	34	ASTM E119
NBSIR 80-2488 (8)	2 x 8	24	5/8" X	Direct	No	54 - 93%	5.9	N/A	24	New T/T

1 I = I-joist; MPCT = Metal Plate Connected Truss; TJL = Trus Joist's Tubular Pin End Connected Truss; FD = Full Design; 1000 max. = 1000 psi max. bending stress (F_b) allowed for the joist.

2 The numbers within parentheses indicate the total test duration

3 Test reports for these data are unavailable

Table 27. Summary of Single Membrane Protected Assembly Tests

4-2.11 Evaluation of Single Membrane Protected Test Assembly Performance

In general, assemblies using lightweight wood components greater than 10 in. in depth and having directly applied 5/8 in. Type C gypsum wallboard have a fire endurance rating greater than 45 min. Resilient channels enhanced this performance by 6 min. (in one test). Also, in one assembly where insulation was applied in the truss cavity (only test summary data are available for this test), the assembly rating was 45 min. with structural failure occurring at 52 min., which means the addition of insulation did not radically alter the fire endurance performance of the assembly. Unfortunately, test reports and results from similar steel component assemblies (e.g., bar joists) are not available. The data do suggest, however, that a 7.5 in. steel C-joist will have fire endurance ratings similar to 12 in. trusses. This would have to be verified with additional testing before any clear conclusions could be drawn.

A single layer of 1/2-in regular gypsum board on lightweight wood components like steel joists and wood trusses contributes fire endurance of 16 and 18 min., respectively, in full-scale room tests (based on two tests). For 2 x 10 joists, 1/2 in. regular gypsum wallboard directly applied contributes 25 min. Based on the Canadian Building Code's Fire Endurance Assembly Calculation Method, if it is assumed that wood trusses contribute 5 minutes, and wood joists contribute 10 minutes of performance to an assembly, the contribution of 1/2 in. regular gypsum wallboard is 13 min. on trusses, and 15 min. on joists.[2] Applying this concept more broadly, it would appear that 1/2 in. of regular gypsum wallboard provides an additional 10 to 15 min. of performance to most lightweight structural members to which it is attached.

It is interesting to note, however, that under high levels of excess air, ASTM E119 predicted the results that were seen in the room fire tests in **Section 4-2.9**. The time/temperature curve based on the room fire tests predicted a far faster time to failure than was seen in the room itself. Unfortunately, it is not believed that enough testing has been done so that room tests are accurately represented by standardized time/temperature curves. Additionally, much of the testing cannot be directly compared due to changes in the test protocol between tests.

[2] Canadian Wood Council, **Wood and Fire Safety**, 1991, p. 123.

Chapter 4-3: Fire Endurance Performance of Connections

Many of the lightweight component assemblies employ some type of connector to connect the smaller-dimension pieces used to form the component. Connection performance under fire conditions is pivotal to the performance of the structural system. The following test report summaries detail fire performance of connectors under specific test conditions:

4-3.1 Report: The Fire Resistance of Metal Connectors

Authors: R.H. Leicester, C.A. Seath and L. Pham

Sponsor: General Research

Date: After 1977 and prior to 1979

Basic Test Description: With little information available on the fire resistance of exposed joints fabricated with metal connectors, these tests were undertaken on typical timber tension joints. The joists were fabricated out of Blackbutt seasoned to 12% moisture content and having a cross section of 1.96 in. by 3.5 in. The four basic types of joints tested were:

- A nailed joint using 36 nails, each with a 0.17 in. diameter, 3.5 in. long.
- Two 14 gauge, metal connector plates, each being 2.95 x 9 in..
- A bolted joint using six 0.47-in. diameter bolts.
- A split-ring connector joint, using four 2.5-in. diameter split rings.

Test Methods Used: Each test joint was placed in the furnace and loaded in tension to the design working load specified in Australian Standard 1720. The furnace temperature was raised in accordance with Australian Standard 1530. Joint extension was recorded. Two tests of each joint were performed.

In a second set of tests, the load on the joints was reduced by 30%. The furnace temperature was controlled to produce a time/temperature relationship intended to simulate the temperatures measured by Rodack and Ingberg for a typical fire in a residential building.

Report Observations: The failure criterion of a joint for this study was a 0.39-in. joint extension. The time for each joint to reach failure was: 4 min. for the metal plate-connected joint, 11 min. for the split-ring joint, 14 min. for the bolted joint, and 33 min. for the nailed joint. In the second test, the bolted and nailed joints did not fail the joint extension criterion. The metal connector plate failed at approximately 24 min. and the split-ring joint failed at approximately 22 min.

Report Conclusions: Of the four types of joints tested, only the nailed joint showed satisfactory fire resistance characteristics. The mechanism for the poor characteristics of the other joints has been identified, and using this information it may be possible to design a joint with good fire resistance characteristics.

> **Comments:** THE DESIGN WORKING LOAD WAS NOT SPECIFIED, NOR ITS CALCULATION REVEALED, OTHER THAN BY REFERENCE TO THE AUSTRALIAN STANDARD 1720, FROM THE AUSTRALIA TIMBER ENGINEERING CODE. IT IS NOT KNOWN WHETHER THE DESIGN JOINTS WERE LOADED TO EQUIVALENT ALLOWABLE STRESSES ON EACH CONNECTOR, OR IF LUMBER WAS AT FULL DESIGN TENSION WORKING STRESS. WITHOUT THIS INFORMATION, IT IS DIFFICULT TO DETERMINE IF THE COMPARATIVE PERFORMANCE OF CONNECTORS BEING EVALUATED WAS ON AN EQUIVALENT STRESS BASIS. NOTE: THE TIME/TEMPERATURE CURVE USED IN TEST 1 IS NEARLY IDENTICAL TO THE ONE USED IN ASTM E119.

4-3.2 Report: The Fire Behavior of Timber in Wood-Based Products

Author: P.E. Jackman

Sponsor: Timber Research and Development Association (TRADA), High Wycombe

Date: 1980

Basic Test Description: Very little is known about the fire behavior of metal connections in conjunction with solid timber elements. The building code in England is, therefore, very conservative in this respect, and recommends that all connections be protected by sacrificial timber: either by burying the connection behind the assumed charring line, or by overcladding with adequate timber. TRADA evaluated the behavior of dense-nailed plywood gussets as a connection technique. A similar test on unprotected tooth plate connectors was performed.

Test Methods Used: The test methods were not delineated in the paper from which this information was taken. The standard time/temperature curve found in British Standard 476, Part A was used.[1]

Report Observations: The plywood gusset did not fail until the thickness was reduced, by charring, to a point where the stress in the gusset was close to the ultimate strength of the cold material.

Report Summary: It was anticipated before the test series that the plywood gusset failure would occur due to the nails losing their fixity from heat conduction into the timber substrate. This did not occur. The nails were still fixed adequately enough to

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[1] It is assumed this time/temperature relationship is similar to ASTM E119.

provide the required shear strength for the joint to remain firm up to the point where the plywood failed.

Unprotected tooth plate connectors were tested under pure tension and failed in under ten minutes under identical test conditions. Since tooth plate connectors are limited to roof construction (which is not required to have fire resistance), their performance was felt to be satisfactory.

Report Conclusion: It can be seen that with the present state of the art, timber can meet all the requirements expected of a modern building material. In some sections of the building industry there is suspicion for the use of combustible materials. Performance of wood products must be proven and reproven, and backed up with necessary education. Wood and wood based products can make an important contribution to fire safe building construction.

> **Comments:** Few specifics are available on the testing that was performed, as this information was taken from a paper already summarizing the results. It indicates that, as in the previous report, nailed connections perform better than truss-plate joints under full design load and E119 conditions. Based on the two previous reports, truss plates appear to have a fire endurance performance of less than 10 min. under standardized test conditions.

4-3.3 Report: Flame Exposure Tests of a Ceramic Covering System for Truss Plate-Connected Wood Members

Authors: Proprietary

Sponsor: Proprietary

Date: July 6 - August 6, 1990

Basic Test Description: Testing was performed using experimental test procedures employing both a control and test specimen. Each lumber piece was cut so that one cut edge would be used to form a joint for both the control and test specimen. Two other two-foot sections attached to the common cut joint were also from the same lumber piece. Lumber selected for joint fabrication was Spruce Pine Fir. To include a broad range of lumber densities, ten pieces each of 2100F-1.8E MSR, 1650 F-1.5E MSR, and visually graded #3 were used in the testing. (MSR stands for Machine Stress Rated lumber, and visual graded means lumber grades were assigned by a human lumber grader.)

The covering that was applied over the metal connector plate for protected specimens was a 3.5 x 4 in. proprietary ceramic covering, 0.040 in. thick (tolerance being greater than 0.000 and less than 0.009 in.). The metal connector plates used were a 3 x 3.5 in. truss plate.

Test Methods Used: A total of 120 specimens were tested: 60 (20 in each grade) with the 1.5 in. face of the lumber exposed to flame, and 60 with the 3.5 in. face of the lumber exposed to flame. Thirty control and thirty covered specimens were tested in each exposure condition. The 3.5 x 4 in. covering was stapled over both truss plates in each test specimen. The splice was centered between supports, and a gas burner was mounted below the splice to produce a 1500° F flame impingement on the specimen. A 40 lb. dead weight was positioned on the centerline of each splice. The test duration consisted of the time elapsed from initial flame exposure until the specimen deflected a distance of 1-3/8 in.

Report Observations:

Horizontal (3.5" Face) Placement Over Flame	**Number Specimens Tested**	**Average Failure Time (min.)**	**Ratio-Test Over Control (%)**
#3 SPF Control	10	2.88	
#3 SPF Test	10	6.00	208
1650F SPF Control	10	3.00	
1650F SPF Test	10	6.47	216
2100F SPF Control	10	3.22	
2100 F SPF Test	10	6.88	214
Vertical (1.5" Face) Placement Over Flame			
#3 SPF Control	10	4.04	
#3 SPF Test	10	7.08	175
1650F SPF Control	10	4.65	
1650F SPF Test	10	8.67	186
2100F SPF Control	10	5.83	
2100 F SPF Test	10	11.27	193
Total Specimens Tested	**120**		

Table 28. Test Results for Proprietary Coating Tests.

Report Summary:

Description	Placement Over Flame	
	Horizontal	**Vertical**
Average Improved Performance over Control	212%	185%
Standard Deviation	0.279	0.182
Coefficient of Variation	0.078	0.033
Maximum Improvement	298%	228%
Minimum Improvement	140%	146%
Estimated 5th Percentile Improvement	165%	156%

Table 29. Summary of Results for Proprietary Coating Tests.

Comments: THE DATA PRESENTED IN THIS TEST WERE USED TO EVALUATE THE PERFORMANCE OF THE TRUSS CONNECTOR PLATE CERAMIC COATING ONLY. THE DATA DO NOT PROVIDE ANY MEASURE OF THE PERFORMANCE OF A METAL TRUSS CONNECTOR PLATE IN AN ACTUAL FIRE ENDURANCE SITUATION. THIS TESTING DOES, HOWEVER, PROVIDE SIGNIFICANT INFORMATION ON THE ABILITY OF A COATING TO PROTECT A METAL CONNECTOR PLATE AND IMPROVE ITS PERFORMANCE IN A FIRE ENDURANCE ENVIRONMENT.

UNFORTUNATELY, THESE DATA CANNOT BE RELATED TO ANY OF THE OTHER DATA ON CONNECTOR PERFORMANCE TESTS.

4-3.4 Report: Fire Behavior of Metal Connectors in Wood Structures

Author: O. Carling

Sponsor: Royal Institute of Technology, Stockholm, Sweden

Date: 1991 International Timber Conference, London, England.

Basic Test Description: Instead of testing a full-scale connection in a traditional fire test, the behavior of a single connector was studied under electrically generated temperatures that relate to fire conditions. The main difference between this and a furnace test is that only the contact surface is exposed to thermal degradation, while the remaining surface is unaffected.

Two bolt diameters—0.47 in. and one which was unspecified—were tested. A prismatic steel plate was also tested, in order to study the influence of the radius of curvature of the contact surface.

Only one type of nail was tested: annular ring shank nails. The nails had a diameter of 0.15 in. Two lengths were tested: 1.57 and 2.63 in. Nail spacing was 1.18 in. for all tests. Nail edge and end distance were greater than 1.57 in.

Test Methods Used: The tests were carried out in a universal press. The wood specimen was fixed to a base plate. Compressive load was applied to the top of the connection. The bolt and the steel plate were electrically heated, and the temperature at the contact surface was continuously recorded. The displacement between the bolt or steel plate and the wood specimen was recorded. Wood specimens were made of Swedish Pine. The average density was 26.21 lb./ft^3. Before testing, the wood specimens were conditioned in a climate providing a moisture content of approximately ten percent. Most boards were sawn from the center of the log, which includes a high percentage of juvenile wood. This may have affected results in an unfavorable way. Three different rates of temperature increase were studied:

- 20° K/min. up to 572° F, then constant.
- 40° K/min. up to 707° F, then constant (667° F for nail tests).
- 60° K/min. up to 842° F, then constant (752° F for nail tests).

There were 270 bolt and 180 nail tests performed. Failure was defined as the moment the rate of displacement exceeded 0.39 in./min., or the total displacement exceeded 0.69 in.

Report Observations: Specific data summaries are not provided in this report.

Report Summary: The following conclusions were made regarding bolted connections:

The rate of displacement depends on:

- The angle between load and grain direction.
- The bolt bearing stress.
- The rate of temperature increase.

The critical temperature may be lower than the normal charring temperature of wood (approximately 572° F). The critical temperature is lower when load is perpendicular to grain than when load is parallel to grain. When all other circumstances are similar, the rate of displacement is higher:

- When load is parallel rather than perpendicular to grain.
- The higher the bolt bearing stress.
- The higher the steel temperature.
- The smaller the bolt diameter.

The time to failure (in minutes) may be estimated by the following expression:

$$t_{cr} = \frac{T_0 - 20}{\Delta T} + \frac{15}{\Delta T}(14 - \sigma)\left(1 - \frac{5}{\Phi}\right)$$

where ΔT = bolt temperature increase (K/min)

$$T_0 = \begin{cases} 280 \text{ when load is parallel to grain} \\ 340 \text{ when load is perpendicular to grain} \end{cases}$$

σ = bolt bearing stress (MPa)

Φ = bolt diameter (mm)

Nailed Connections

The following conclusions can be made for nailed connections:

- The rate of displacement is higher when load is perpendicular to grain than when it is parallel to grain.
- The rate of displacement is higher for connections made with 1.57-in. long nails than with 2.63-in. long nails.
- When all other conditions are equal, the time to failure is approximately the same, whether load is parallel or perpendicular to grain.
- Within the maximum test time (approximately 25 min.), failure was achieved only for connections with 1.57-in. long nails.

The time to failure (in minutes) for 1.57-in. long nails may be estimated with the following expression:

$$t_{cr} = \begin{cases} \left(7.5 - 0.04\sqrt{30F - 5700}\right)^2 & \text{for } \Delta T = 60 \text{ K/min.} \\ \left(10.3 - 0.055\sqrt{30F - 6500}\right)^2 & \text{for } \Delta T = 40 \text{ K/min.} \\ > 25 & \text{for } \Delta T = 20 \text{ K/min.} \end{cases}$$

Where ΔT = steel-plate temperature increase (K/min.)

F = nail load (Newtons per nail)

Comments: THESE TESTS WERE DEVELOPED SPECIFICALLY FOR THE REFERENCED PROJECT. THEY WERE NOT BASED ON STANDARDIZED TEST PROCEDURES. FORMULA VERIFICATION HAS NOT BE PERFORMED.

4-3.5 Report: The Fire Performance of Unloaded Nail-On Gusset Connections For Fire Rated Timber Members

Authors: P.K.A. Yiu and A.B. King

Sponsor: Building Research Association of New Zealand (BRANZ)

Date: 1988

Basic Test Summary—Unloaded tests: The starting point for this study was a series of fire tests of unloaded nailed connections. Blocks of glue-laminated timber had steel and plywood gusset plates nailed to one side. The gussets were then protected with gypsum plaster board, solid timber, or intumescent paint before exposure to a standard fire in the BRANZ pilot furnace. Temperatures were measured between the various layers during the test. It was found that steel gussets with no protection had a rapid rise in temperature, leading to charring around the gussets and around the nails. Plywood gussets with no protection charred layer by layer—much faster than would be expected for solid wood. Solid wood and gypsum plaster board gave good protection to both steel and plywood gussets, with a slow but steady temperature rise in the gusset. Intumescent paint gave some protection, with a faster temperature rise than for gypsum plaster board.

> **Comments:** These tests provide general information about protection of connections using coverings. These data reinforce results from the proprietary test discussed earlier.

4-3.6 Report: Behavior of Nailed Gusset Connections Under Simulated Fire Exposure

Authors: A.H. Buchanan, R. Chinniah and P.J. Moss

Sponsor: Building Research Association of New Zealand

Date: 1988

Test Methods Used: Steel and plywood gussets were nailed to blocks of glue-laminated timber and loaded in shear. A horizontal orientation was used in order to get uniform heating over the gusset plate. This required a pulley system to transfer the horizontal load to the vertical action of the testing machine. A heating box with a domestic electric stove heating element and thermostat was used to raise the temperature of the gusset plates. Temperatures in the test specimens were measured with thermocouples. The time/temperature curves followed were from the unloaded BRANZ test, and were non-standardized.[2]

Basic Test Description: Glue-laminated radiata pine timber 3.54 in. thick was used for all tests. Moisture content was in. the range of 10 - 13%. For the steel gusset plates, plain steel nails—2.95 in. long by 0.124 in.—were used for most tests. Galvanized nails—1.18 and 1.51 in. by 0.124 in. and 1.77 by 0.17—were also used. Nail heads were driven to just making contact with the steel gusset in order to eliminate friction effects. For the plywood gussets, 3.34 x 0.13-in. gun nails were used. The steel gussets were 0.19-in. thick mild steel plate, pre-drilled with six holes. The plywood gussets were

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[2] Yiu, P.K.A. and King, A.B., 1989, "Fire Performance of Unloaded Nail-on Gusset Connections for Fire Rated Timber Members," Draft Study Report, Building Research Association.

0.71-in. thick construction plywood. All gussets were nailed to the glue-laminated timber with six nails.

Three types of protection were tested: one layer of 0.75-in. Fyreline Gibraltar Board, two layers of 0.57-in. Fyrestop Gibraltar Board, and intumescent paint. Both cold tests under increasing load, and constant load tests with increasing temperature were performed.

Report Observations: Ultimate load and load slip curves were recorded for each test performed.

Report Summary: Under simulated fire conditions, the nail slip in steel plate gusset connections increased with increasing load, and also with increasing gusset temperature. For plywood gussets, only one thickness of protection was simulated. Nail slip increased with increasing load. Measured nail slips for plywood gussets were generally larger than for steel gussets with the same protection.

For the same protection (one layer of 0.75-in. Fyreline Gibraltar Board) a nail load of 86.5 lbs. (1.8 times the basic nail load for 0.124-in. diameter nails), the slip after one hour of exposure was 0.03 in. for the steel gusset and 0.08 in. for the plywood gussets. The ultimate load capacity of nails was approximately ten times the basic nail load under cold conditions, and six times the basic nail load after one hour of simulated fire exposure.

> **Comments:** THE VALUE OF PROTECTION FOR CONNECTION SYSTEMS IS SHOWN IN IMPROVED FIRE PERFORMANCE, WHICH WAS ALSO NOTED IN THE PREVIOUS TWO REPORTS.

4-3.7 Report: Bolted Steel Plate Joints in Timber Structures Under Fire Conditions

Authors: O. Holmijoki, J. Majamaa and E. Mikkola

Sponsor: Fire Technology Laboratory, VTT, Finland

Date: 1991 International Timber Engineering Conference

Basic Test Description: Ignition and charring models for wood were applied to mechanical bolted steel plate joints of timber structures. A calculation method was developed to determine the rate of temperature increase of the steel plate, and the time to charring conditions in wood under different fire exposures. Experimental results using Cone Calorimeter tests gave the charring rate values under the steel plate and effect of bolts on charring.

Test Methods Used: The tests were carried out using the Cone Calorimeter equipment of the Fire Technology Laboratory of the Technical Research Center of Finland. All wood specimens were the same size: 3.93 x 3.93 x 2.99 in. Specimens were wrapped in a thin aluminum foil (except the upper surface), and the lower edge was insulated using fire resistant Kaowool insulation blanket. Three specimens were tested. The first was a single block of laminated veneer lumber. The second was a block of laminated veneer lumber with a steel plate covering the top of the specimen. The third was a block of laminated veneer lumber with only a steel bolt penetrating the specimen from the top to the bottom.

Test specimens were exposed to a constant heat flux of 25, 50 or 75 kW/m^2. The temperature in the specimens was measured by thermocouples located on the surface, and at various depths within the specimen. The time to ignition and the rate of mass loss were also measured. The duration of any single test was 30 to 60 min.

Report Observations: Test results were collected, and values for time for charring to start (t_i) were calculated. Here, d is the thickness of the steel plate.

Test Specimen	**q (cone)** (kW/m^2)	**d** (mm)	**t_i (Test)** (min:sec)	**t_i (Calc.)** (min:sec)
Wood	25	—	1:48	2:11
	50	—	0:15	0:15
	75	—	0:07	0:06
Wood & Steel Plate Covering (no bolt)	25	8	13:20	13:05
	50	5	3:40	3:57
	50	8	5:20	5:59
	50	12	8:20	8:37
	75	8	4:20	3:50

Table 30. Test Results for Fire Technology Laboratory.

In the steel bolt test, charring of the wood was observed to be slower in contact and near the steel bolt than elsewhere in the wood specimen. At a heat flux level of 50 kW/m^2, it took thirty minutes before heat transfer through the steel bolt caused a higher charring rate near the bolt than elsewhere in the specimen.

Report Summary: As shown by the test results, a steel plate covering a wood specimen causes protection of the wood by delaying the initiation of charring, and reducing the charring rate, as compared to free burning situations(those without steel plates). This effect is closely related to the thickness of the steel plate. Increasing the thickness of the steel plate increases the protection effect. On the basis of this study, it can be stated that the charring rate of wood under steel plate joints can be calculated from the rate of mass loss and temperatures.

> **Comments:** THIS REPORT APPEARS TO CONFIRM THE THEORY THAT STEEL GUSSET PLATES CAN PROTECT WOOD FROM CHARRING AS RAPIDLY AS UNPROTECTED WOOD. RESULTS ALSO SHOW THAT STEEL BOLTS PENETRATING WOOD DO NOT CAUSE THE SURROUNDING WOOD TO CHAR MORE RAPIDLY.

THESE RESULTS MIGHT APPEAR TO CONTRADICT LOGIC AND, THEREFORE, MAY BE DIFFICULT TO ACCEPT. ADDITIONAL WORK NEEDS TO BE DONE TO DETERMINE IF THIS PHENOMENON HAS ANY MEANINGFUL EFFECT ON THE FIRE PERFORMANCE OF AN ASSEMBLY, AND IF IMPROVEMENTS IN PERFORMANCE CAN BE MADE USING THESE CONCEPTS.

4-3.8 Report: None Yet Available

Authors: R.H. White, S.M. Cramer, R.W. Wolf

Sponsor: National Forest Products Association

Date: Committee on Research and Evaluation (CORE) Report, April 4, 1991

Basic Test Description: Testing was performed on metal connector plates. The plates were 20 gauge, Grade A steel with 9.4 teeth/in^2. Each tooth was approximately 0.3 in. long. Two types of plates were used in the testing: one with teeth slots parallel to the grain of lumber, measuring 2.95 x 7.5 in., having 96 teeth; the other with teeth slots perpendicular to the grain of lumber, measuring 2.96 x 6.88 in. and having 91 teeth. Joints were made using two plates with teeth removed from the middle 0.66-in. of the plate over the butt joint. The lumber used was #1 DNS (Dense) Southern Pine Visual Grade and 2100F-1.8E SPF MSR.

Test Methods Used: There were two test methods used: one called "ramp load to failure," and the other, "fire exposure to failure". Under ramp load, a constant temperature was placed on the specimen. The temperatures used were: room temperature, 100, 200, 250, 275, 300, and 325° C. Exposure times were typically 30 min., but some 60 min. exposures were made. Under fire exposure to failure, the ASTM E119 time/temperature curve was used. The test specimens were stressed to 50- and 100% of design load. Pure tension and tension moment stresses were placed on the specimen. These data will be used to verify the thermal degrade model.

Report Observations: The final report on these data has not been completed.

Comments: THE DATA DEVELOPED FROM THIS STUDY WILL PROVIDE THE GREATEST AMOUNT OF INFORMATION TO DATE ON THE FIRE PERFORMANCE OF TRUSS CONNECTOR PLATES. THESE DATA ARE BEING USED TO REFINE A COMPUTER MODEL THAT WILL PREDICT THE FIRE ENDURANCE PERFORMANCE OF A SINGLE TRUSS. FROM THIS, THE METHODOLOGY WILL BE EXPANDED TO MODEL THE FIRE PERFORMANCE OF AN ENTIRE TRUSS ASSEMBLY.

Connector Test Summary Table

Testing Designation	Structural Member	Connection Type	Test Temperature	Load Condition	Average Time To Failure	Comments
Aus. Tests I	2 x 4	3 x 9 MPC**	ASTM E119	Design working load	4 min.	0.39" Jt. Ext. = Failure
Aus. Tests I	2 x 4	4-2.5" Split Rings	ASTM E119	Design working load	11 min.	0.39" Jt. Ext. = Failure
Aus. Tests I	2 x 4	6 - .5" Bolts	ASTM E119	Design working load	14 min.	0.39" Jt. Ext. = Failure
Aus. Tests I	2 x 4	36-.17x3.5" long nails	ASTM E119	Design working load	33 min.	0.39" Jt. Ext. = Failure
Aus. Test II	2 x 4	3 x 9 MPC	Res. time/temp	70% Design level	approx. 24 min.	0.39" Jt. Ext. = Failure
Aus. Test II	2 x 4	4 - 2.5" split rings	Res. time/temp	70% Design level	approx. 22 min.	0.39" Jt. Ext. = Failure
Aus. Test II	2 x 4	6 - .5" bolts	Res. time/temp	70% Design level	> 125 min.	0.39" Jt. Ext. = Failure
Aus. Test II	2 x 4	36 - .17"x3.5" long nails	Res. time/temp	70% Design level	> 125 min.	0.39" Jt. Ext. = Failure
TRADA Test	2 x 4	MPC's	BS 476	Tension loads	< 10 min.	All Data Not Available
TRADA Test	2 x 4	Dense Nailed Ply. Gusset	BS 476	Tension Loads	N/A	All Data Not Available
Finnish Tests	3.9 x 3.9 x 3"	Wood Only	25 kW/m2	none	1.8 min.	Time to Start of Charring = Failure
Finnish Tests	3.9 x 3.9 x 3"	Wood Only	50 kW/m2	none	.25 min.	Time to Start of Charring = Failure
Finnish Tests	3.9 x 3.9 x 3"	Wood Only	75 kW/m2	none	.11 min.	Time to Start of Charring = Failure
Finnish Tests	3.9 x 3.9 x 3"	.5 in. Steel Plate Cover	25 kW/m2	none	13.33 min.	Time to Start of Charring = Failure
Finnish Tests	3.9 x 3.9 x 3"	.19 in. Steel Plate Cover	50 kW/m^2	none	3.6 min.	Time to Start of Charring = Failure
Finnish Tests	3.9 x 3.9 x 3"	.3 in. Steel Plate Cover	50 kW/m^2	none	5.3 min.	Time to Start of Charring = Failure
Finnish Tests	3.9 x 3.9 x 3"	.47 in. Steel Plate Cover	50 kW/m^2	none	8.3 min.	Time to Start of Charring = Failure
Finnish Tests	3.9 x 3.9 x 3"	.3 in. Steel Plate Cover	75 kW/m^2	none	4.33 min.	Time to Start of Charring = Failure
FPL Tests	N/A	2.95 x 7.5 in., 96 Teeth MPC	Constant Temp	ramp load	N/A	Data Not Yet Available
FPL Tests	N/A	2.95 x 6.88 in., 91 Teeth MPC	ASTM E119	50 & 100% design load	N/A	Data Not Yet Available
Proprietary*	2 x 4	3 x 3.5 MPC	1520 ° F	40 lbs.	3.06 min.	avg. 4x2 unprot. control
Proprietary*	2 x 4	3 x 3.5 MPC	1520 ° F	40 lbs.	4.86 min.	avg. 4x2 unprot. control
Proprietary*	2 x 4	3 x 3.5 MPC	1520 ° F	40 lbs.	6.43 min.	avg. 4x2 prot. test
Proprietary*	2 x 4	3 x 3.5 MPC	1520 ° F	40 lbs.	9.00 min.	avg. 4x2 prot. test

* Proprietary tests are protected (prot.) control vs. protected (prot.) tests under load and fire.

** MPC = Metal Plate Connector, Ply. = plywood.

Table 31. Summary Table of Connection Fire Endurance Tests.

4-3.9 Evaluation of Connection Fire Endurance Testing Performance

Connection systems form a critical element in any structural system. In all likelihood, the connection will fail prior to any other element of a structural member under most loading schemes, including those due to fire degradation. The Australian tests raise concerns about metal plate and split ring connections when compared to bolts and nails. However, it is not known if these tests were run under equivalent connection stresses. The TRADA test reinforces the data available on metal plate connectors. The proprietary metal plate connector tests show only the effect of coatings, and do not represent connection failure times for comparative purposes. The FPL tests are the most comprehensive tests performed on metal plate connectors and should provide the basis for performing more significant analyses. The Finnish tests reinforce the concept that steel coverings can protect wood prior to conduction and wood charring. The char rate is shown to be lower under steel plates and adjacent to steel bolts.

At this time, not enough information on connection system fire endurance performance is available to allow broad-based conclusions to be made about their impact on structural system performance under fire exposure. The FPL testing should provide this information, and the model developed should allow for better prediction of the performance of metal plate connected truss systems.

Chapter 4-4: Fire Endurance Performance of Operation Breakthrough Assemblies

Operation Breakthrough was initiated by the Department of Housing and Urban Development (HUD) in May 1969 to demonstrate industrialized housing techniques that could be used for high volume production. As part of its research, HUD conducted numerous fire tests of assemblies to determine and demonstrate fire performance. The following summarizes the tests and results obtained when these assemblies were evaluated.

4-4.1 Report: Feedback - Operation Breakthrough, Volume 5, Part 3, Fire Endurance: Roofs/Ceiling, Floor/Ceiling and Floor Assemblies

Author: United Stated Department of Housing and Urban Development

Sponsor: United Stated Department of Housing and Urban Development

Date: Initiated in May, 1969

Test Method Used: All tests were performed using the ASTM E119 time/temperature curve. The loading used was specific to the test being performed.

Reference: Fire test reports RC-168, RC-169, and RC-171, National Gypsum Company Research Center, January and February 1972 (Unpublished).

Report Summary:

Double Wood Joists, Plywood and Gypsum Board System: The ceiling assembly consisted of 2 x 4 wood joists spaced 16 in. on center, with one layer of 1/2 in. Type X gypsum wallboard and one layer of 3 1/2 in. thick fiberglass insulation between the joists. The end of the joists were nailed to a double 2 x 6 edge beam, on top of which was built a 16 ft. x 16 in. high parapet wall, constructed with 2 x 4 studs, 24 in. on center. Both sides of the stud wall were sheathed with 1/2 in. plywood. The roof assembly consisted of 2 x 6 wood joists, 16 in. on center, nailed to 2 x 6 edge beams. The roof sheathing material was 1/2 in. plywood.

Testing: Three tests were conducted on three separate test assemblies. Each assembly measured 11 ft., 9 1/2 in. x 17 ft., 5 in.

Test 1: Described above.

Test 2: This test was identical to Test 1 except that the ceiling assembly was insulated with two layers of 3 1/2 in. thick glass fiber batts instead of one. A 1/4 in. bead of adhesive was applied to each joist before the wallboard was nailed.

Test 3: This test was identical to Test 2 except that an additional layer of 1/2 in. Type X gypsum board was added to the ceiling surface.

Each test had a superimposed load of 30 lbs./ft^2 applied to the roof joists. The ceiling joists had dead load applied only.

Test Results:

Test 1: Structural collapse occurred at 34 min., 30 sec.

Test 2: At 45 min., 10 sec., excessive temperature rise was recorded at one thermocouple. Flamethrough followed at 45 min., 20 sec.

Test 3: At 83 min., 40 sec., flamethrough occurred on the unexposed side of the roof system.

Reference: Son, B.C., "Fire Endurance Test of a Roof/Ceiling Construction of Paper, Honeycomb and Gypsum Board," NBSIR 73-167, National Bureau of Standards, January, 1973

Test Description: This roof/ceiling assembly consisted of two panels: each 8 ft., 11 in. wide and 13 ft., 5 in. long, butted together on the long sides to produce a test panel 13 ft., 5 in. by 17 ft., 10 in. The nominal overall thickness of the assembly was 7 1/4 in. The sandwich panels consisted of flame retardant treated paper honeycomb core with 5/8 in. Type X Gypsum on both sides. The edge of each honeycomb core had 3 x 6 in. beams consisting of 4 layers of 3/4 in. plywood. A 5 in. wide strip of 5/8 in. Type X gypsum board covered the joint between the test panels. A uniform load of 15.9 lbs./ft^2 was applied to the test specimen. This produced a bending moment equivalent to that produced by a conventional design load of 20 psi over a 12 ft. span.

Test Results: Failure occurred at a corrected time of 37 min., 13 sec., by flamethrough of the unexposed surface through a joint in the gypsum boards. At 37 min., 23 sec., a local load failure occurred.

Reference: Fire Test Report FC-156, National Gypsum Company (Unpublished)

Test Description: Construction of this assembly was identical to that of Report NBSIR 73-167, above, except that the gypsum board strip covering the panel joint on the exposed side was 6 in. wide instead of 5 in. Type C wallboard was used instead of Type X on the fire exposed side of the panel. A uniform load of 17 lbs./ft^2 was applied during the test.

Test Results: Flamethrough at the panel joint occurred at 29 min.

4-4.2 Reference: Fire Test Report FC-159, National Gypsum Company (Unpublished)

Test Description: The sandwich panel was produced identically to that in NBSIR 73-167 except that two layers of 5/8 in. Type C gypsum board were applied to the ceiling side of the roof system. Type C board was used instead of Type X on the roof side as well. The exposed layer was bonded to the under layer with 3/16 in. beads of adhesive spaced 12 in. on center, and stapled with 1 1/2 in. long staples. Staples were spaced 24 in. on center along each edge and down the center of each board, and 12 in. on center at end joints. A uniform load of 18.5 lbs./ft^2 was applied to the test specimen.

Test Results: Flamethrough at a gypsum board joint on the unexposed surface occurred at 64 min., 45 sec.

Reference: Son, B.C., "Fire Endurance Test of Steel Sandwich Panel - Exterior Wall and Roof/Ceiling Constructions," NBSIR 73-135.

Test Description: The construction of this roof/ceiling system consisted of a 3 in. thick paper honeycomb core partially filled with solid polyurethane foam, and 26 gauge sheet steel facings on both sides. The test assembly consisted of four 4 ft. x 13 ft., 5 in. deep sandwich panels and one 1 ft., 10 in. x 13 ft., 5 in. deep panel. Long edges of the panel were closed with a 1-1/2 x 5-1/4 in. tongue-and-groove wood closure. A 26 gauge, galvanized sheet metal cap was applied over the top of this closure. Overall dimensions of the test assemblies were: 13 ft., 5 in. x 17 ft., 10 in. A uniform load of 28.6 lbs./ft^2 was applied during the test, which is equivalent to 40 lbs./ft^2 over a 12 ft. span.

Test Results: A maximum temperature rise of 325° F occurred at one thermocouple on the unexposed side at 9 min., 9 sec.

Reference: Report Number 5067, "Standard ASTM Fire Endurance Test on a Roof and Ceiling Assembly," Building Research Laboratory, Ohio State University (Unpublished)

Test Description: The test specimen was composed of 0.151 in. thick glass fiber reinforced polyester skins bonded to the top and bottom of truss type stiffeners made of the same material. The cavities formed by the stiffeners were filled with proprietary insulation material. 2 x 6 wood rim joists provided a surround for the nominal 6 in. thick roof panel. The stiffeners and external surfaces of the rim joists were coated with an intumescent paint. The test specimen, 11 ft., 7-1/2 in. x 16 ft., was loaded at eight load points to a uniform live load of 20 lbs./ft^2.

Test Results: The test assembly could not sustain the applied load after 48 min. of exposure to fire.

4-4.3 Reference: Project 5234, "Report of a Standard ASTM Fire Endurance Test of a Limited Load Bearing Roof and Ceiling Assembly," Building Research Laboratory, Ohio State University, March, 1972 (Unpublished).

Test Description: The roof/ceiling system consisted of 20 gauge, galvanized sheet steel, interlocking pans, 4 in. deep x 16 in. wide x 12 ft., 5 in. long. The pans were installed with their vertical legs up. Unfaced, 3 1/2 in. thick glass fiber insulation batts were placed in the recesses formed by the vertical legs, and a 1 in. thick rigid glass fiber insulation was installed over the entire assembly. Roof sheathing was 1/2 in. exterior grade plywood. The ceiling consisted of 1/2 in. Type X gypsum board, which was attached to steel furring channels, 24 in. on center, perpendicular to the steel pans. A 12 ft., 5 in. x 16 ft. test specimen was loaded to produce a uniform load of 30 lbs./ft^2 over an 11 ft., 11 in. clear span.

Test Results: After 42 min. of exposure to fire, hydraulic load jacks were no longer able to apply load due to the deflection of the specimen. When the test was terminated at 47 min., the system had deflected more than 8 in. No flamethrough was observed, nor were any excessive temperature rises recorded on the unexposed surface.

4-4.4 Reference: "Report on a Fire Endurance Test of a Floor and Ceiling Construction," UL File R6946-1, Underwriters' Laboratories, Incorporated, February, 1972 (Unpublished).

Test Description: The floor consisted of 7-1/2 in. steel C-joists, spaced 24 in. on center, covered with a 3/4 in. tongue-and-groove interior grade plywood underlayment. One-half of the floor was covered with pad and shag carpet, the other half with 1/16 in. vinyl asbestos tiles. One layer of 1/2 in. Type SF-3 gypsum board was attached directly to the bottom flanges of the joist. The ceiling consisted of 1/2 in. Type SF-3 gypsum board attached to 7/16 in. deep steel furring channels, spaced 12 in. on center, running perpendicular to the joist span. A 12 ft., 5 in. x 16 ft., 6 in. assembly was loaded with 45 lbs./ft^2.

Test Results: At 52 min., flamethrough occurred on half of the exposed floor surface that was covered with vinyl floor tile. This was followed by structural collapse at 52 min., 45 sec.

Reference: Fire Test Report FC-170, National Gypsum Company Research Center, February, 1972 (Unpublished)

Test Description: The floor system consisted of 2 x 8 wood joists, spaced 16 in. on center, with 5/8 in. plywood subflooring. The ceiling system was made up of 2 x 4 joists, spaced 16 in. on center, with one layer of 5/8 in. Type X gypsum board applied to the bottom side of the joist. Paper-faced, 3 1/2 in. glass fiber insulation bats were installed between the ceiling joists. A uniform load of 40 lbs./ft^2 was applied to the 10 ft., 10 1/2 in. x 17 ft., 5 in. test assembly.

Test Results: Flamethrough occurred on the unexposed side of the joists at 45 min., 30 sec.

Reference: Fire Test Report FC-166 and FC-167, National Gypsum Company Research Center, December, 1971 and January, 1972 (Unpublished - Tests 1 and 2 below); and Son, B.C., "Fire Endurance Tests of Plywood and Steel Joist Floor Assemblies Without Ceilings," NBSIR 73-141, March, 1973 (Test 3 below).

Test Description:

Test 1: (FC-166) The floor system consisted of 6 in. deep, 18 gauge galvanized steel C-joists, spaced 24 in. on center, with 3/4 in. tongue-and-groove plywood subflooring attached to the joists. The ceiling assembly was composed of 3 in. deep, 18 gauge galvanized steel C-joists, spaced 24 in. on center. The ceiling membrane consisted of 3/8 in. plywood attached to the underside of the steel joists. 5/8 in. Type C gypsum board was applied over the 3/8 in. plywood. Glass fiber blanket insulation, 2 in. thick, was laid over the top of the ceiling joists. A 40 lbs./ft^2 load was applied to an 11 ft., 8 in. x 17 ft., 4 in. specimen.

Test 2: (FC-167) This test was identical to Test 1 except that the ceiling membrane consisted of two layers of 1/2 in. Type C gypsum board.

Test 3: (NBSIR 73-141) This test was similar to Tests 1 and 2 except that it was slightly larger in size: 11 ft., 9 in. x 17 ft., 11 in. The ceiling membrane was a single layer of 5/8 in. Type X gypsum board and a continuous 3 in. wide, 24 gauge steel bracing strap, which was welded to the top of the ceiling joists at mid span.

Test Results:

Test 1: Flamethrough occurred at 50 min.

Test 2: This test was terminated at 70 min., 30 sec., when structural failure appeared imminent.

Test 3: Failure occurred at 30 min. by flamethrough on the unexposed floor surface.

4-4.5 Reference: Son, B.C., "Fire Endurance Test of a Steel Sandwich Panel Floor Construction," NBSIR 73-164, National Bureau of Standards, April, 1973.

Test Description: The structural frame of the floor assembly consisted of 6 x 3 in., 14 gauge steel C-joists. The joists were spaced 48 in. on center. The overall size of the assembly was 10 ft., 7 1/4 in. x 17 ft., 11 in. The steel C-joists were unsheathed.

Sandwich panels were placed over the C-joists. The sandwich panels were 3 in. thick, paper honeycomb core, with a top surface of 3/8 in., CD interior grade plywood, and a bottom surface of 26 gauge, galvanized sheet steel. Three 4 ft. wide panels were placed in. the center of the test assembly with one 2 ft., 11 1/2 in. panel on either side of these, for a total of five panels. Joints between the panels were sealed with 3/8 in. wide Butyl sealant strips. A 40 lbs./ft^2 load was applied to the floor assembly.

Test Results: Failure by flamethrough occurred at a joint between two sandwich panels at 8 min., 45 sec., followed by structural failure at 9 min.

Test Number	Structural Member	Spacing	Gypsum Type	Ceiling Application	Insulation System Used	Applied Load (psf)	Assembly (min:sec)
RC-168 NG	2x4 joists / 2x6 rafters	16" o.c.	1/2" X	Direct	3 1/2" glass	30	34:30
RC-169 NG	2x4 joists / 2x6 rafters	16" o.c.	1/2" X	Direct	7" glass	30	45:20
RC-171 NG	2x4 joists / 2x6 rafters	16" o.c.	2-1/2" X	Direct	7" glass	30	83:40
NBSIR 73-167	3x6 beams (4 lyr. 3/4" plywd.)	8'11"	5/8" X	Direct	Honeycomb core	15.9	37:13
FC-156 NG	3x6 beams (4 lyr. 3/4" plywd.)	8'11"	5/8" C	Direct	Honeycomb core	17	29:00
FC-159 NG	3x6 beams (4 lyr. 3/4" plywd.)	8'11"	2-5/8" C	Direct	Honeycomb core	18.5	64:45
NBSIR 73-135	Steel-faced paper honeycomb	48" o.c.	none	26 ga. steel faces	honeycomb foam	28.6	9:09
BRL 5067	Glass Fiber/ Polyester Panel	16' 0"	none	0.151" poly. skin	proprietary	20	48:00
BRL 5243	20 ga. steel pans	16" o.c.	1/2" X	R/C Channel	glass	30	47:00
UL R6946-1	7 1/2" steel joists	24" o.c.	2-1/2" SF-3	R/C Channel	none	45	52:45
FC-170 NG	2x8 / 2x4 joists	16" o.c.	5/8"	Direct	glass	40	45:30
FC-166	6" 18 ga C / 3" 18 ga C steel	24" o.c.	3/8" ply./5/8"X	Direct	glass	40	50:00
FC-167	6" 18 ga C / 3" 18 ga C steel	24" o.c.	2-1/2" C	Direct	glass	40	70:30
NBSIR 73-141	6" 18 ga C / 3" 18 ga C steel	24" o.c.	5/8" X	Direct	glass	40	30:00
NBSIR 73-164	3x6 14 ga C-joists / sndw. pan.	48" o.c.	none	26 ga. steel faces	Honeycomb	40	8:45

Table 32. Summary of Operation Breakthrough Reports

4-4.6 Evaluation of Fire Testing of Operation Breakthrough Assemblies

Operation Breakthrough testing produced a wide variety of information with very little standardization upon which to make comparisons. In the two fire tests (NBSIR 73-135 & 73-164) that used unsheathed steel joists or a steel sandwich panel, the results were similar to other unsheathed tests (described in **Chapter 4-1**) with the assembly lasting under 10 min. in both cases. When the steel joists were protected by gypsum wallboard, the fire performance increased commensurate with the protection. It should be noted that in the test that used two layers of 1/2 in. Type C gypsum board, the performance of the steel joist assembly was 70 min., 30 sec. It is generally assumed that virtually any system that uses two layers of Type X or Type C gypsum board attached directly to the bottom of the component will result in a fire endurance time greater than 60 min. Based on available test reports, this is applicable for engineered wood assemblies, steel joists, and 2 x 4/2 x 6 joist rafter assemblies as well.

Unfortunately, it is extremely difficult to make more than general observations about the HUD tests, as not enough is known about the specifics of each. However, this information can be used to provide additional data on some of the performance characteristics of a variety of assemblies and effects such as gypsum board type, combinations of gypsum board, glass fiber insulation, and the use of resilient channels. Any further reliance on these data would be unrealistic.

Chapter 4-5: Fire Endurance Performance of Coatings

The literature search did not produce significant information on the fire endurance performance of coatings, as that was not its focus. The information included in this chapter provides data for discussion and illustrates the potential for improving fire performance of lightweight products through use of coatings.

4-5.1 Report: Flame Exposure Tests of a Ceramic Covering System with Truss Plate Connected Wood Members

Authors: Proprietary

Sponsor: Proprietary

Date: July 6 - August 6, 1990

Basic Test Description and Test Method Used, See **Chapter 4-3: Fire Endurance Performance of Connections**.

Report Observations:

Horizontal Placement (3.5" Face) Over Flame	**Number Specimens Tested**	**Average Failure Time (min.)**	**Ratio Test Over Control (%)**
#3 SPF Control	10	2.88	
#3 SPF Test	10	6.00	208
1650F SPF Control	10	3.00	
1650F SPF Test	10	6.47	216
2100F SPF Control	10	3.22	
2100 F SPF Test	10	6.88	214
Vertical Placement (1.5" Face) Over Flame			
#3 SPF Control	10	4.04	
#3 SPF Test	10	7.08	175
1650F SPF Control	10	4.65	
1650F SPF Test	10	8.67	186
2100F SPF Control	10	5.83	
2100 F SPF Test	10	11.27	193
Total Specimens Tested	**120**		

Table 33. Results of Proprietary Covering Tests.

These data from the table above are represented graphically in Figures 31 and 32:

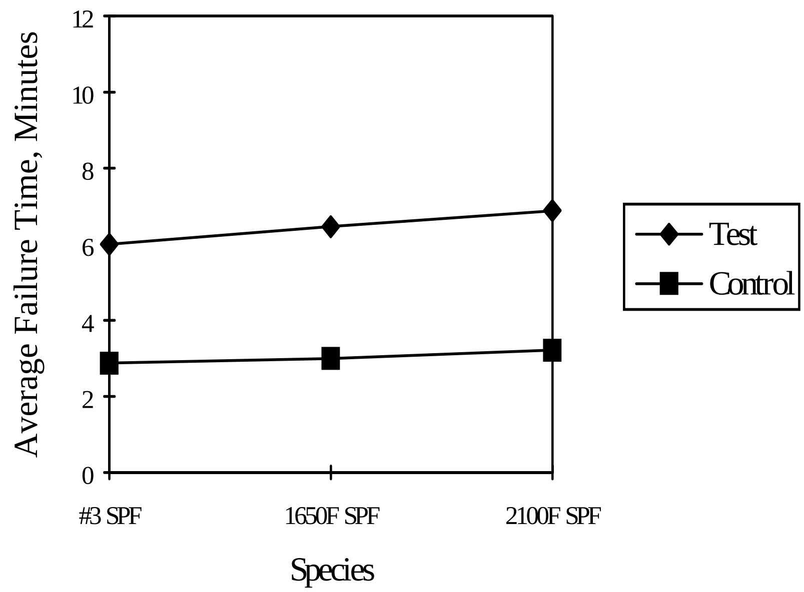

Figure 31.   Average Failure Time for Horizontal Specimens

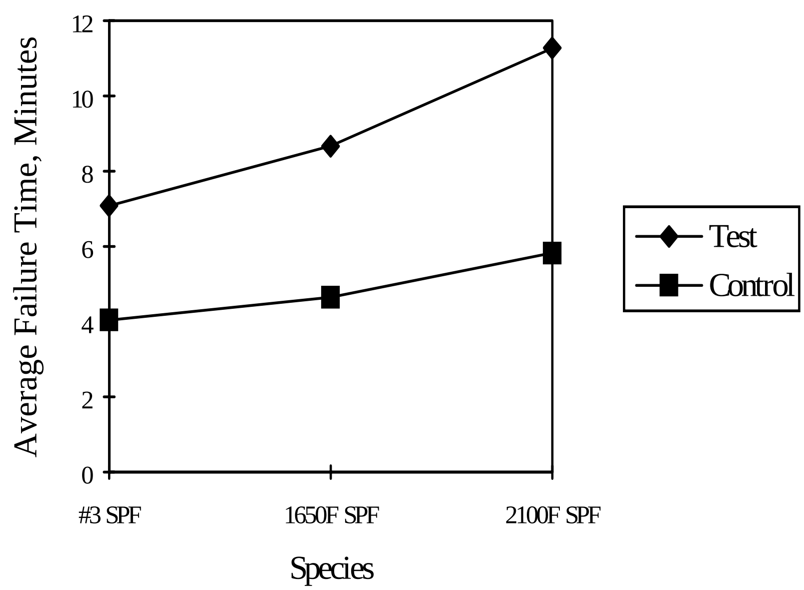

Figure 32.   Average Failure Time for Vertical Specimens

Report Summary:

Description	Placement Over Flame	
	Horizontal	**Vertical**
Average Improved Performance over Control	212%	185%
Standard Deviation	0.279	0.182
Coefficient of Variation	0.078	0.033
Maximum Improvement	298%	228%
Minimum Improvement	140%	146%
Estimated 5th Percentile Improvement	165%	156%

Table 34. Summary of Results for Proprietary Coating Tests.

Comments: THE FIRE DATA PRESENTED IN THIS TESTING INDICATES THAT PROTECTION OF THE CONNECTOR IMPROVES THE FIRE PERFORMANCE OF THE CONNECTION ASSEMBLY. THE CONCEPT OF IMPROVED CONNECTION PERFORMANCE USING A HEAT RESISTANT COVERING OR COATING CAN PROBABLY BE SUCCESSFULLY APPLIED TO MOST CONNECTION TYPES. IF AN ADEQUATE COVERING OR COATING IS APPLIED TO THE CONNECTION, ONE CAN THEREFORE ANTICIPATE IMPROVEMENT IN FIRE ENDURANCE PERFORMANCE.

4-5.2 Report Fireball Tests of Open Webbed Steel Joists

Author: T.E. Waterman, IIT Research Institute

Sponsor: General Services Administration

Date: May 15, 1977

Basic Test Description and Test Method Used: These are identical to that found in **Chapter 4-1: Fire Endurance Performance of Unsheathed Assemblies in Section 4-1.18**.

Report Observations: Unsheathed Joists generally reach temperatures between 1400 and 1600° F when exposed to the Factory Mutual Research Corporation Test F time/temperature curve.

Test 1: None of the unsheathed joists in this test met the temperature limitation of 1100° F.

Test 2: These were joists protected with two coats of intumescent paint: Each joist was brush painted with one coat of Pratt and Lambert primer at a rate of 700 ft.^{2}/gal., followed by two coats of Pratt and Lambert fire retardant white paint (intumescent type) at a rate per coat of 200 ft.^{2}/gal. On each joist, the upper chord nearly met the 1100° F limit, except where ceiling corrugations exposed unsheathed surfaces. Webs and lower chords all exceeded the limit, but were kept cooler than the unsheathed joists of Test 1.

In general, the unprotected surface temperatures ranged from 1400 to 1600° F. The intumescent coated surfaces ranged from 1200 to 1400° F.

Test 3: These joists were protected with four coats of intumescent paint: The joists were prepared with one coat of primer and four coats of intumescent paint as described in Test 2. The intumescent coats were inadvertently brushed out thinner than before, so that the four coats were equivalent to three coats of the recommended thickness of 200 ft.^{2}/gal. Improvement in protection was marginal, at best. Temperatures were more erratic than before and, in some cases, poor performance resulted. It is suspected that the benefit of additional coating thickness was nullified by coating losses during intumescence. Further consideration of brushed on intumescent coatings was abandoned.

Test 4: These were joists protected with Jet-Sulation Type 400—a commercial sprayed fiber fireproofing. Experienced applicators coated the joist using a low pressure gun. The applicators were instructed to put on the thinnest coat practical. It is estimated that 1/4 in. of material would be adequate, but it was found difficult to apply such a thin coating.

Adequate protection was achieved in all cases except for one round lower chord, where the applied coating was less than 1/16 in. thick. Thickness ranged from 1/4 in. to 1 3/4 in. of Jet-Sulation.

Test 5: Albi-Clad 89-S—an intumescent fireproofing mastic—was professionally applied to the joists using heavy-duty pneumatic spray equipment. A wet film thickness of 1/16 in. was applied to Joists A, C and D; 1/8 in. to Joists B and E. During the early stages of the fire exposure, the coating intumesced to a thickness of approximately 1 in. Some flaming near the coating was noted as it intumesced, and it appeared to separate from the joists in localized areas. Neither thickness of Albi-Clad offered adequate protection for the joists, and little difference in protection was observable between the two applied thicknesses.

Test 6: Joists protected with Cafco products: Two coatings were professionally applied for this test. Two joists were coated with Cafco Blaze-Shield D C/F, and three others with Cafco Deck-Shield C/F. Both coatings are insulative in nature. Blaze-Shield is described as mineral fibers in a cementitious binder, designed for application to rigid structural assemblies. Deck-Shield is described as cementitious in nature, designed for roof and wall assemblies. Both are applied with a low pressure gun which incorporates a small amount of water during application. The applicators were instructed to apply the thinnest coat practical. Upper chords were protected by the applied coatings, suggesting that as little as 1/4 in. is adequate at the ceiling. A coating thicknesses of 1/4 in. was adequate in protecting all webs and the round lower chord. Coating thickness of approximately 1/2 in. were necessary to protect the thinnest lower chords, with lesser amounts needed for thicker sections.

Report Conclusions: Based on this testing, the use of spray-on fireproofing insulation appears to be the best coating to protect joists in GSA Records buildings from fire temperatures achieved prior to sprinkler operation. Coating thicknesses near the minimum compatible with commercial application techniques appear suitable. Since these coatings could significantly reduce heat losses if applied to the entire ceiling, GSA might consider this combined benefit by coating the entire ceiling and support system. It is probable that coatings of greater insulative quality and lesser fire resistance would also offer the fire resistance necessary for this purpose, while increasing energy savings.

A summary of this testing is presented in the following table:

Test Number	Structural Member	Spacing (in o.c.)	Treatment	Comments
IITRI J6397 1	12" deep Joist A-E	12	none	Failed; joist temp>1000° F FM "F" curve
IITRI J6397 2	12" deep Joist A-E	12	2 coats intum. paint 200 ft.^{2}/gal.	Failed; joist temp>1000° F FM "F" curve
IITRI J6397 3	12" deep Joist A-E	12	4 coats intum. paint 200 ft.^{2}/gal.	Improvement marginal over #2
IITRI J6397 4	12" deep Joist A, D	12	Jet-Sulation Type 400	Met 1100° F criter., i.e, FM "F" curve
IITRI J6397 5	12" deep Joist B-E	12	Albi-Clad 89-S, 1/8" & 1/16"	Failed; joist temp>1100°F, FM "F" curve
IITRI J6397 6	12" deep Joist A-E	12	Cafco Blaze-Shield D C/F	Passed, 1/4" for TC* webs, 1/2" for BC*
IITRI J6397 6	12" deep Joist A-E	12	Cafco Blaze-Shield C/F	Passed, 1/4" for TC* webs, 1/2" for BC*

* TC = Top Chord; BC = Bottom Chord

Table 35. Results of Fireball Tests.

Comments: AGAIN, COATINGS ARE SHOWN TO PROVIDE IMPROVED FIRE PERFORMANCE. THE IMPROVEMENT REALIZED IS DIRECTLY DEPENDENT ON COATING TYPE USED AND ITS APPLICATION. THIS DATA IS PRESENTED FOR GENERAL INFORMATION ONLY.

4-5.3 Evaluation of Fire Testing of Coatings

Information on coating performance for fire protection of structural members is very limited. Much of the testing has been done on structural steel columns with spray-applied fire insulation or the application of wallboard. These protection systems have been applied to structural steel beam and joist elements as well. Calculation methods have been developed for this protection.[1] Proprietary testing has also been performed using the ASTM E119 test to provide the details necessary for specific coating materials to meet a variety of ratings.[2] Most of this testing has been done to facilitate regulatory acceptance of structural systems where a fire resistance of one hour or greater is specified.

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[1] See Chapter 31 of the Southern Building Code and Uniform Building Code Standard 43-9 for model code methods of calculating fire resistance of structural assemblies.

[2] ASTM E119 ratings for steel also have temperature limits, as noted in **Chapter 4**.

Available tests and calculation procedures indicate that coatings already have a place in fire protection for structural elements. More data are needed to evaluate how coatings may best be used. It can be expected that the greater use of coatings will be dependent upon economics and the establishment of performance requirements for the intended application. Without these criteria, the full potential of coatings may not be easily realized.

Chapter 5: Sprinkler Testing

5.1 Overview

In **Quick Response Sprinklers: A Technical Analysis**,[1] it is revealed that the primary focal point of sprinkler testing to date has been on sprinkler performance and their ability to contain a fire under a variety of fire load conditions. Little attention has been placed on structural elements holding up the sprinklers. However, one of the current questions, from a fire service perspective, is what will be the performance of lightweight engineered wood products with sprinklers attached after a fire has begun. The key elements of this question are:

- Will structural members fail before the sprinklers can activate to control the fire or protect the structure?
- Does the arrangement of the sprinklers and their attachment to the structural element effect a sprinkler's ability to protect the structure?
- Does the spacing, depth, and arrangement of the structural element influence the sprinklers' ability to protect the structure?

This chapter will report on the available test data specific to fire performance of lightweight engineered components supporting sprinklers. Unfortunately, data on this specific issue is limited. The test reports found in the literature search are on wood systems exclusively.

5.2 **Report:** Fireball tests of Open-Webbed Steel Joists.

Author: T.E. Waterman, IIT Research Institute

Sponsor: General Services Administration

Date: May 15, 1977

Report Details: See **Section 4-1.18**of **Chapter 4-1** for report details.

> **Comments:** THE FIREBALL TESTS OF OPEN WEBBED STEEL JOISTS IN **Chapter 4-1** ALSO DISCUSSED THE PERFORMANCE OF STEEL BAR JOISTS PRIOR TO SPRINKLER ACTIVATION. THE CRITERION USED FOR ACCEPTABLE PERFORMANCE WAS PREVENTION OF TEMPERATURES GREATER THAN 1100° F

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[1] Fleming, Russell P. "Quick Response Sprinklers: A Technical Analysis", National Fire Protection Research Foundation, April 1985.

ANYWHERE ON THE STEEL JOISTS. THIS TEST IS INCLUDED FOR GENERAL INFORMATION.

5.3 **Report:** Sprinkler Tests for Protection of Parallel Chord Wood Trusses

Authors: D. Burkhart, K. Powell, and R.B. Coker

Sponsor: Fort Worth Fire Prevention Bureau

Date: 1988

Basic Test Description: The City of Fort Worth investigated sprinkler coverage in concealed truss spaces. Trusses tested were 16 in. deep wood webbed, metal plate connected trusses. The test assembly consisted of a 30 x 30 ft. area that had trusses spaced 2 ft. on center. A 3/4 in. tongue-and-groove plywood deck was used as sheathing. The sprinkler system consisted of a tree system with a 2 in. cross main and riser nipple, and 1-1/2 in. branch lines to reduce friction loss. Outlets were arranged to allow for installation of 8 x 8, 10 x 10, and 12 x 12 ft. spacing (standard and staggered), both in the upright and pendent positions. Horizontal distances from truss members were random.

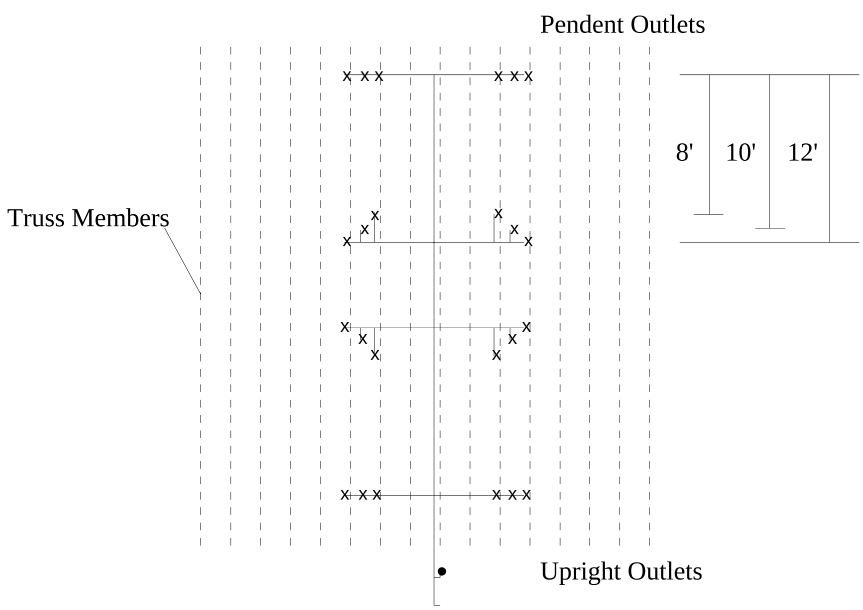

Figure 33. Test Assembly Layout

Distribution Criteria: The criteria established for successful distribution were wetting of the lower chord members in the entire sprinkler effectiveness area, and some wetting of the deck, established by visual inspection.

Test Results: The first test series used 12 x 12 ft. spacing with standard sprinklers installed in standard positions. Tests were performed using operating pressures of 7, 15, and 25 lbs./in.^{2}. The established criteria were not met.

The next series of tests were the same as Test 1, but used standard sprinklers installed in their reverse position. This meant that upright sprinklers were installed in pendent position and the pendent sprinklers were installed in the upright position. This resulted in excellent wetting of the deck, but not of truss members.

The third test reduced spacing to 10 x 10 ft. With standard sprinklers installed in normal positions, the established criteria were not met. The criteria were also not met when standard sprinklers were installed in reverse positions.

The final series of tests was run with standard sprinklers and staggered 10 x 10 ft. spacing. Satisfactory results occurred with standard sprinklers in reverse positions, and an operating pressure of 7 lbs./in.^{2}. Satisfactory results also occurred with conventional sprinklers in upright and pendent positions, at an operating pressure of 7 lbs./in.^{2}.

Fire Tests: The same assemblies for the distribution tests were used in the fire tests, except that in fire tests a 1/2 in. standard drywall ceiling was attached to the bottom chords of the trusses. The sides and ends were enclosed to limit air circulation in the concealed space. The sprinkler system was charged with water, and the control valve was closed. Upon activation of the first sprinkler, the valve was opened to the desired pressure. Although this caused a slight delay, pressures were more accurately controlled. The pass/fail criterion was total extinguishment. The fuel used for each test was 16 ounces of naphtha placed in an 18 in. diameter pan 2 in. deep. This amount of fuel was used as the ignition source for the wood truss only, not as fuel to contribute to burning.

Test Results: The first test used 10 x 10 ft. staggered sprinkler spacing. The fuel was located in the same channel as the sprinkler, but in the geometric center between the three sprinklers. The test used pendent sprinklers in the upright position. The fire was extinguished within 10 sec. One sprinkler activated, with an operating pressure of 7 lbs./in.^{2}.

In the next series of tests, conventional sprinklers were used in the pendent and upright positions. The fires were also extinguished in less than 10 sec., with a single sprinkler activating with 7 lbs./in.^{2} water flow.

Two tests were also performed with the fire located in a channel adjacent to the sprinklers. In all cases, the fire was extinguished in less than 10 sec. with one sprinkler activating with 7 lbs./in.^{2} water flow.

Finally, fire tests performed with standard sprinklers in normal positions failed to extinguish the fires with water pressures less than 25 lbs./in.^{2}.

Report Summary: The acceptable arrangements were:

- Standard sprinklers staggered at 10 x 10 ft., installed in the reverse position with a minimum operating water pressure of 7 lbs./in.^{2}
- Conventional sprinklers installed either upright or pendent, staggered at 10 x 10 ft., with a minimum operating water pressure of 7 lbs./in.^{2}

Report Conclusion: A proposal was made to amend the local fire code to designate the spacing of sprinklers in parallel chord wood truss construction to a maximum 10 x 10 ft. staggered spacing, with standard sprinklers in the reverse position, or conventional sprinklers in either position, operating at a minimum of 7 lbs./in.^{2} water flow.

> **Comments:** The Fort Worth Tests were performed to provide information about sprinkler distribution patterns within a truss concealed space. This testing was undertaken to determine whether sprinkler head spacing and positioning designated in National Fire Protection Association's NFIPA 13 standard was satisfactory to extinguish a concealed space fire within wood trusses.[2] The report does not state whether or not there are other viable options for sprinkler placement, positioning, or the use of different sprinkler heads that would allow other spacing combinations. Additional testing is desirable. It would also be desirable to have a test procedure with objective and quantitative criteria to evaluate sprinkler system spacing within the concealed space. Without these criteria, performance acceptability is undefined.
>
> These data were submitted to the NFIPA 13 committee. The committee chose not to change the standard believing that the criterion for extinguishment of fire was excessive. Given this, there appears to be the de facto establishment of at least one performance criterion for sprinkler tests—*control* of the fire. The committee did, however, change the definition of trusses.
>
> The City of Fort Worth fire code was changed, adding the following sprinkler placement requirements and revised truss definition (Note: these requirements do not apply to all situations, and are not to be used as general sprinkler placement requirements. See **Chapter 7, Section 7.3.6** for additional discussion.):

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[2] NFiPA 13R, **Standard for the Installation of Sprinkler Systems in Residential Occupancies up to Four Stories in Height**, 1989 ed.

- WHEN WOOD TRUSSES ARE PRESENT IN CONCEALED SPACES, SPRINKLERS SHALL BE PLACED AT A MAXIMUM PROTECTION AREA OF 100 FT.^{2}.
- SPRINKLER HEADS SHALL BE PLACED NOT CLOSER THAN 6 FT., NOR FARTHER THAN 10 FT. FROM ADJACENT HEADS.
- SPRINKLER HEADS SHALL NOT BE CLOSER THAN 6 IN. TO TRUSS MEMBERS.
- SPRINKLER HEADS SHALL BE INSTALLED IN THE REVERSE POSITION—PENDANT SPRINKLERS IN UPRIGHT POSITION OR UPRIGHT SPRINKLERS IN PENDANT POSITION.
- WOOD TRUSS CONSTRUCTION SHALL MEAN PARALLEL WOOD CHORD BEAMS WITH WOOD WEBBING SUPPORTING A ROOF OR FLOOR DECK. TRUSSES WITH STEEL WEBBING SIMILAR TO BAR JOIST CONSTRUCTION HAVING WOOD CHORDS SHALL BE CONSIDERED AS COMBUSTIBLE BAR JOIST CONSTRUCTION.

5.4 Report: Fire Sprinklers in Exposed Deep Prefabricated Wood I-Joists Floor/Roof Systems

Authors: J. Piscione and J. Vogt

Sponsor Trus Joist Corporation and Willamette Industries

Date: February, 1989

Basic Test Description: Three separate fire tests were conducted:

Test 1: The floor assembly consisted of five 24 in. deep TJI 350X I-joists and five 24 in. deep WSI 424 I-joists spaced 32 in. on center and spanning 30 ft. The ends of the joists were attached to a rim beam, which provided firestopping, at the end of each channel for the full depth of the joists. The length, depth and spacing of the joists resulted in a volume of 160 ft.^{3} per joist channel. This volume represents the maximum allowed by the Insurance Services Organization (ISO). Six Viking Micromatic Model M standard response glass bulb spray sprinkler heads, with 1/2 in. orifices, were mounted on two parallel branch lines, located approximately 6 ft., 11-1/2 in. on either side of the assembly centerline. These sprinkler heads have a nominal sprinkler temperature rating of 155° F, and a response time index of 300. The sprinklers were spaced at 9 ft., 4 in. intervals along each branch line, and provided a protection area per sprinkler of 130 ft.^{2}. The spacing was chosen according to the maximum per head protection area allowed in NFIPA 13 for an ordinary hazard occupancy classification. Sprinklers 1 and 6 were located at the center of a joist channel, 2 and 5 directly beneath an I-joist, and 3 and 4 located 16 in. beyond the floor assembly. All sprinkler head deflectors were 25 in. beneath the assembly deck.

An inactive sprinkler branch line was also installed under the floor assembly, approximately 6 in. from one of the active branch lines. Five standard response and two quick response sprinklers were attached to the inactive line. This line was positioned 18 in. beneath the deck. This allowed comparisons to be made between the response of the sprinklers positioned at 18 in., and those at 25 in. beneath the deck.

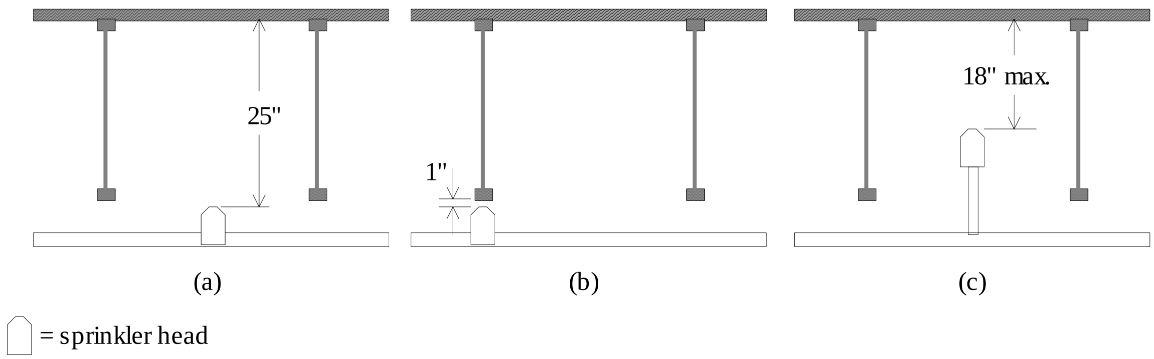

Figure 34. Active sprinkler head placement in test assembly: at center of joist channel (a) and one inch below bottom flange (b). Placement of inactive standard response sprinkler heads at 18 inches beneath the deck (c).

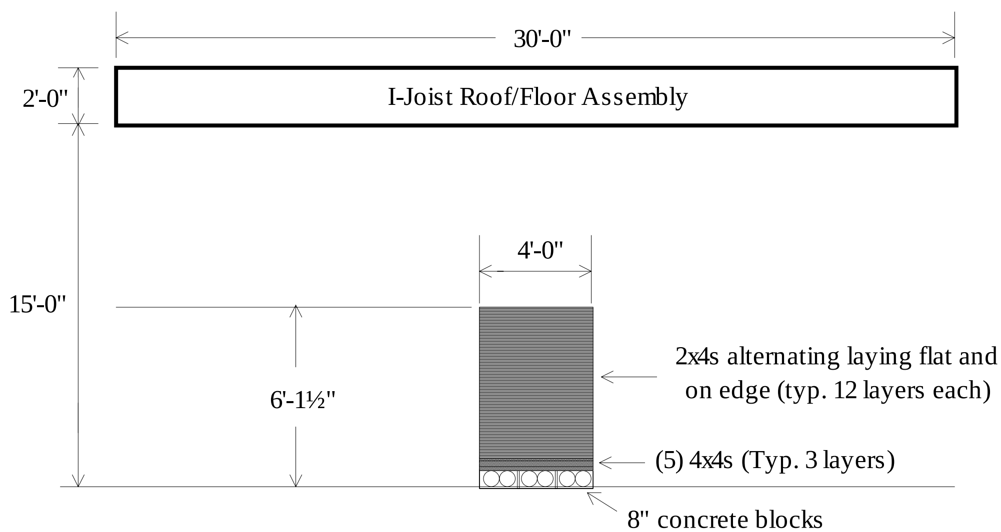

Figure 35. Cross-section of wood crib and floor assembly.

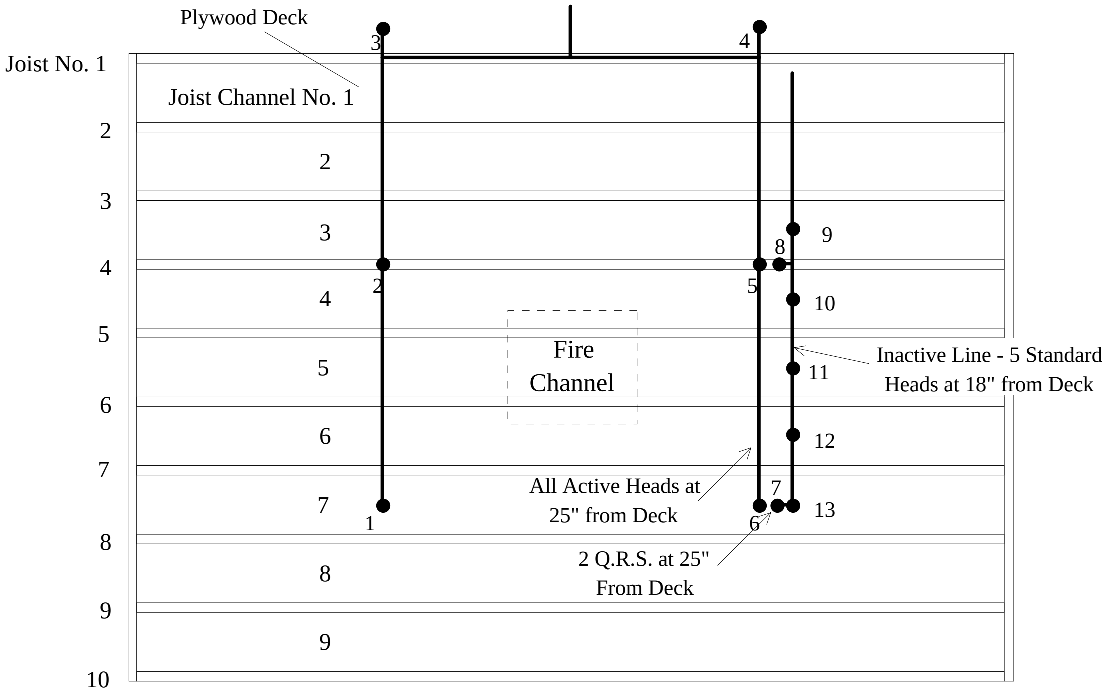

Figure 36. Diagram of Test 1.

Test 2: The second fire test was the same as the first, except that the spacing of the branch lines was at 15 ft., and active sprinkler heads were 10 ft., 8 in. apart. This represented the maximum spacing allowed (130 ft.^{2} per head) in NFIPA13 for ordinary hazard occupancies. ('Ordinary' indicates certain construction types and combustible contents using lightweight engineered construction.)

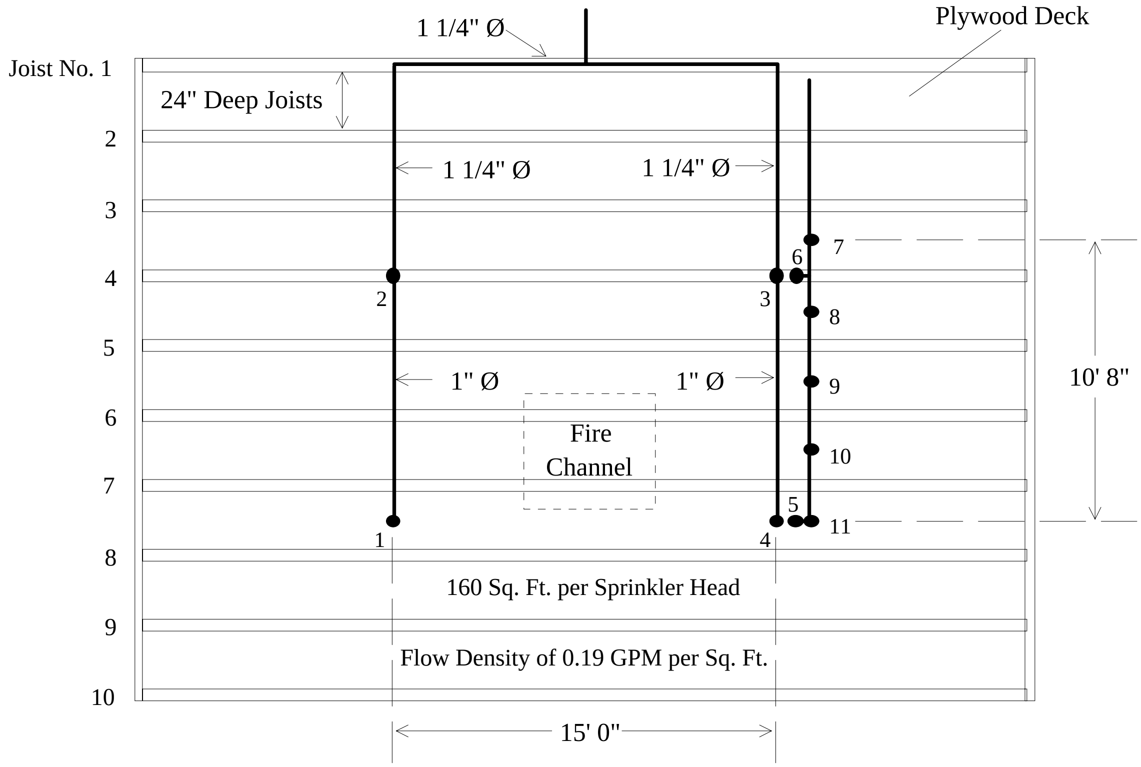

Figure 37.   Diagram of Test 2.

Test 3: This test was identical to the second, except that the effective joist depth was increased to 30 in. All active sprinkler heads were centered between the joists, with deflectors positioned 31 in. beneath the deck.

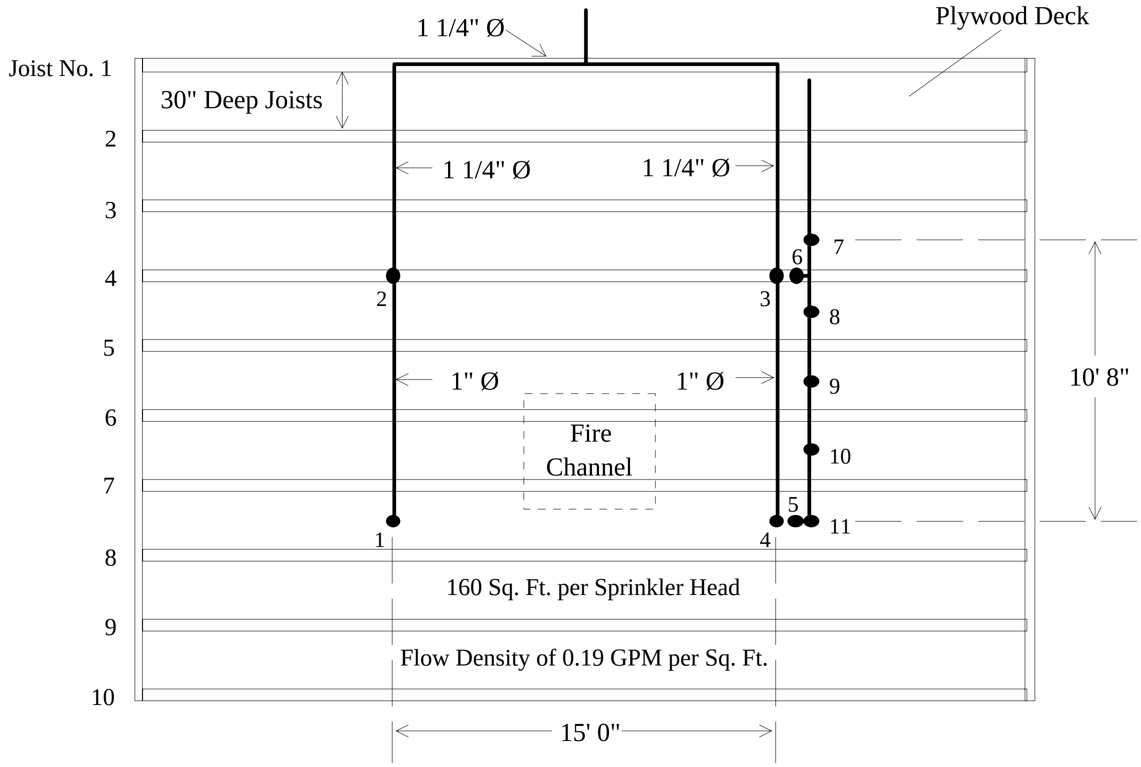

Figure 38. Diagram of Test 3.

Test Methods used: Water supply for the sprinkler system was provided by a pumping system capable of delivering 1680 gal./min. All pipe sizes used in this sprinkler system were selected in accordance with NFIPA 13 specifications. The flow density per sprinkler head during the first test was calculated to be 0.35 gal./min/ft.^{2}. This was reduced to 0.19 gal./min/ft.^{2} for Tests 2 and 3.

The fire source used in each test was provided by a wood crib containing 528 bd.-ft. of dimension lumber. The crib design was similar to that used by Underwriters Laboratories to evaluate the effectiveness of sprinkler heads, but contained over 3-1/2 times the amount of wood. The crib used in Test 1 was positioned directly under joists at the center of the assembly, while the cribs used in Tests 2 and 3 were positioned under joist 6 and 7 in the center of the assembly. The crib placement in each test was considered to be in the worst location with respect to active sprinkler head sensitivity.

Report Observations: Temperature measurements were made in various locations throughout the assembly. Observations were also made during each test.

Report Summary:

Test 1: The exposed surface of the deck at the center of the assembly began to burn in less than 2 min. from crib ignition. The first sprinklers to respond were the uncharged standard response sprinklers 18 in. below the deck at 1 min., 54 sec., and 1 min., 57 sec. An uncharged quick response sprinkler was the next to respond at

2 min., 7 sec., and was located 25 in. below the deck. One more uncharged standard response sprinkler 18 in. below the deck responded before the charged sprinklers. The first charged active response sprinkler activated 2 min., 26 sec. into the test. Within one second, the second standard response sprinkler activated. At 2 min., 34 sec., all fire in the assembly had been completely extinguished. The flames were confined to the wood crib 4 min., 10 sec. into the test, and by 5 min., 40 sec., the fire had been completely extinguished.

Damage to the floor assembly was minimal, and was confined to the center joist channel only. Charring, 1/16 in. thick, was observed on the deck and I-joists of the center channel to a distance of approximately 2 ft. on either side of center. Beyond this distance damage was limited to surface charring and discoloration.

Test 2: This test progressed in a manner similar to Test 1. Three uncharged standard response sprinklers 18 in. below the deck, and two uncharged quick response sprinklers 25 in. below the deck responded before any of the charged sprinklers.

The response time of the charged sprinklers in this test was approximately 3 min., 26 sec. into the test--one minute longer than the response times of the charged sprinklers in Test 1. However, the charged sprinklers in Test 2 were located approximately 50 percent further away from the fire. At approximately 4 min. from activation of the two standard response sprinklers, the fire in the floor assembly had been completely extinguished. At 4 min., 30 sec., flames were confined to the wood crib. By 8 min., 30 sec., the fire was completely out, and the test concluded. Damage to the floor assembly was similar to that observed in Test 1. Charring of approximately 1/16 and 1/8 in. was observed on the I-joists and the exposed surface of the deck on the portion of the joist channel which was located directly over the wood crib. Some charring was noted beyond this area, but was limited to surface charring. Minor discoloration was also noted.

Test 3: As in the two previous tests, two uncharged quick response sprinklers and three uncharged standard response sprinklers activated before any of the charged sprinklers.

Two charged standard response sprinklers activated at 4 min., 10 sec. and 4 min., 11 sec. into the test. These were soon followed by two more activations at 4 min., 15 sec. and 4 min., 21 sec., respectively. At 4 min., 31 sec., all fire in the floor assembly had been extinguished. At 13 min., 56 sec., the fire in the wood crib was completely out, and the test concluded. Damage to the floor assembly during Test 3 was minimal, and confined to that portion of the joist channel located directly above the wood crib. Damage to the assembly beyond the central portion of this channel was limited to surface discoloration.

Report Findings:

- The sprinkler coverages and flow densities used in each test were effective in controlling and extinguishing the fire.

- Temperatures in the I-joist channels located away from the fire were generally higher at sprinklers beneath the joist than at sprinklers inside the joist channel. Sprinkler head sensitivity would thus be optimized by placing the sprinklers below the joists.
- Temperatures appear to be the same at a given elevation and horizontal distance from the fire source. Sprinkler heads, therefore, can be placed directly beneath the I-joists or in the center of the joist channel without jeopardizing sensitivity. The temperature drop in each successive joist channel away from the fire is significant. Therefore, in a larger assembly with a greater number of sprinklers, it is not likely additional sprinklers would be activated.
- Joist channels should be blocked to a maximum of 200 ft.^{3}, which is based on a 30 in. deep joist channel.

Comments: This testing was ad hoc, as there was no defined test procedure, performance requirements or performance criteria. The only criterion for acceptable performance seems to be that the fire is controlled and/or extinguished. Even though temperature measurements were made throughout the assemblies, there is no pass/fail criteria for a temperature rise in any location on an assembly. Previously we have seen that 1100° F was the criterion used for determining the acceptable performance of a steel joist. The Fort Worth test also used fire extinguishment as the only criterion for success or failure of the system. This area needs further development.

5.5 Report: Fire Sprinklers in Exposed 30 in. Deep Prefabricated I-joist Floor/Roof Systems, Phase 2

Authors: J. Piscione and P. Pintar

Sponsors: Trus Joist Corporation and Willamette Industries

Date: November, 1989

Basic Test Description: Six separate tests were conducted in Phase 2. Four tests were reported in detail.

The floor/roof assembly consisted of fourteen 30 in. deep TJI/350X I-joists and twelve 30 in. deep WSI-424 I-joists spaced 32 in. on center. Thirteen joists spanned a 30 ft. section of the structure. The remaining joists spanned a 45 ft. section, creating an overall structure 75 ft., 7 in. in length. The rim joists provided firestopping around the perimeter of the joists. Additional firestops were placed 15 ft. from the outer edge of the 45 ft. joists. The length, depth and spacing of the joist channels resulted in a volume of 200 ft.^{3} per joist channel. Plywood 5/8 in. thick was used for the deck.

Branch lines were located 7 ft., 9-1/2 in. from the ends of the assembly, and spaced 15 ft. apart. Sprinkler heads had a nominal sprinkler temperature rating of 155° F, and a response time index of 300. The sprinklers were spaced at 8 ft., 8 in. intervals along each branch line, and provided a protection area per sprinkler of 130 ft.^{2}. The branch line elevations were adjusted so that all sprinkler head deflectors were 34 in. beneath the deck of the assembly for Test 2, and 31 in. below the deck for Tests 1-2, 3 and 4.

An inactive branch line was also installed approximately 6 in. laterally from each active branch line. Quick response sprinklers were placed on the inactive branch line, that had a rating of 155° F and a response time index of 65.

Test 1-2: The fire source was located on the center-most sprinkler line, between two sprinkler heads. This location was selected in response to a concern that the critical fire location was between two heads, close to a firestop.

No standard response sprinklers were used; only six quick response sprinkler heads were allowed to activate during the test.

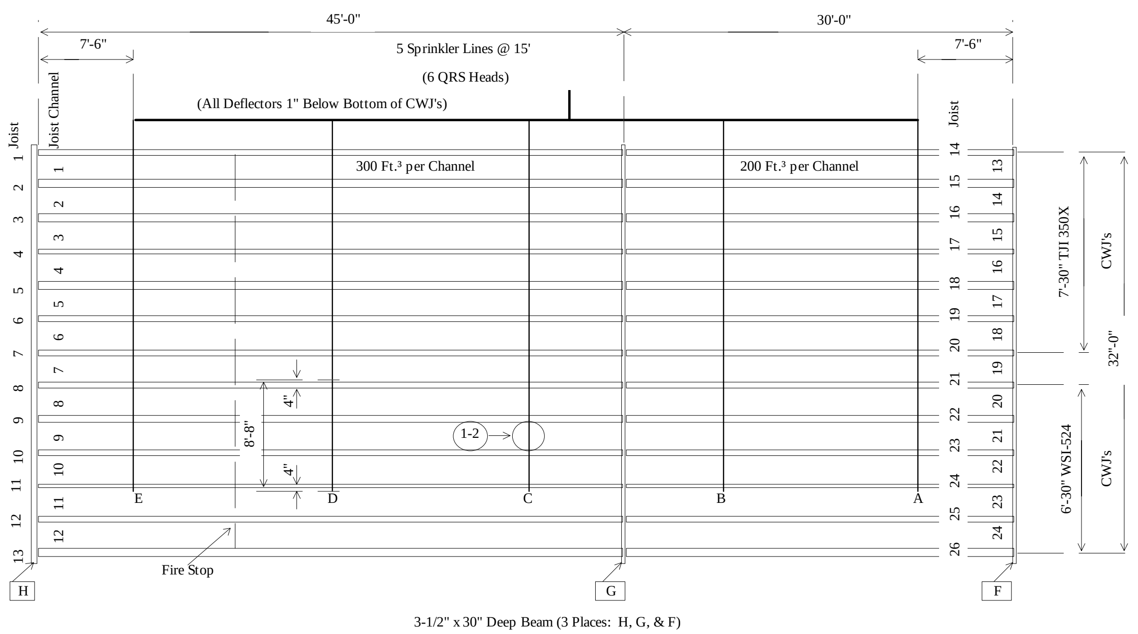

Figure 39. Plan View of Test 1-2

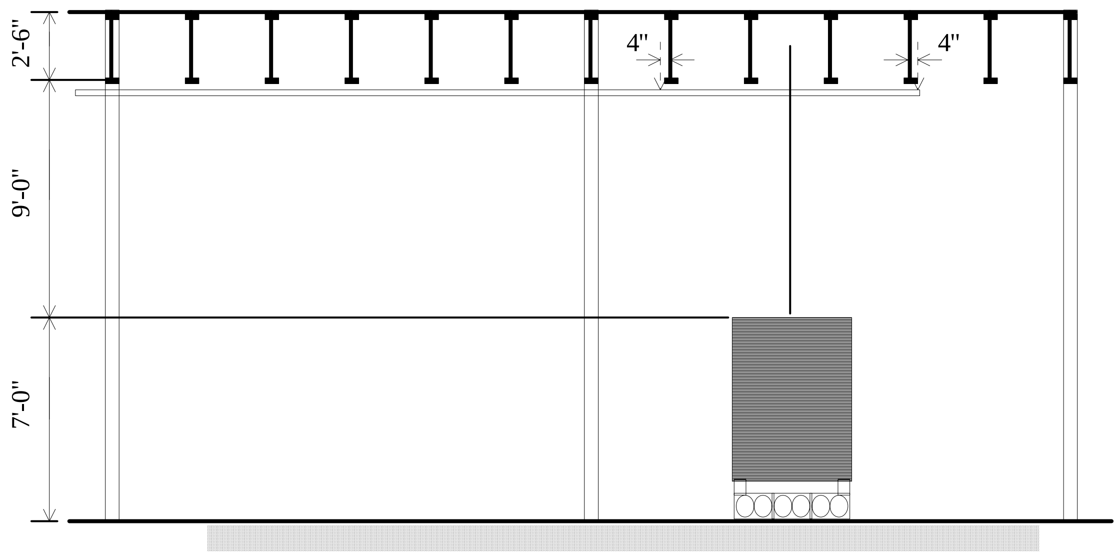

Figure 40.   Elevation View of Test 1-2

Test 2: The fire source was located under the center-most sprinkler head of the branch line next to the center beam under the 30 ft. joists. This fire location was selected in response to a concern that the critical fire location was beneath one sprinkler. Active standard response sprinklers were used. The sprinkler head deflectors were located 34 in. below the bottom of the deck. Fifteen standard response and fifteen quick response sprinklers were deployed.

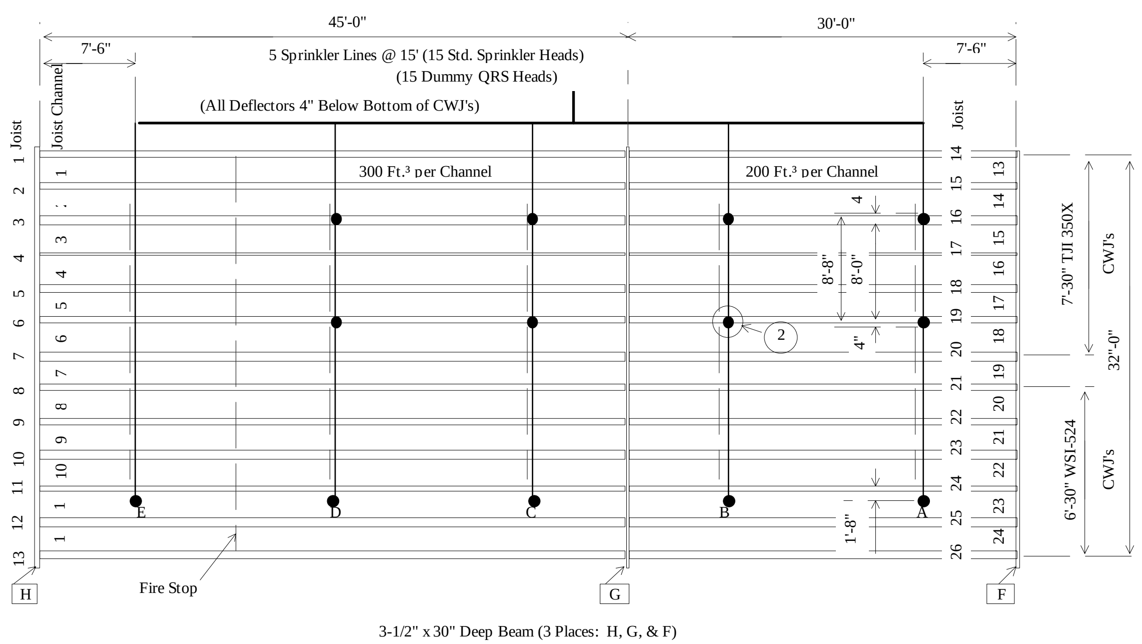

Figure 41.   Plan View of Test 2

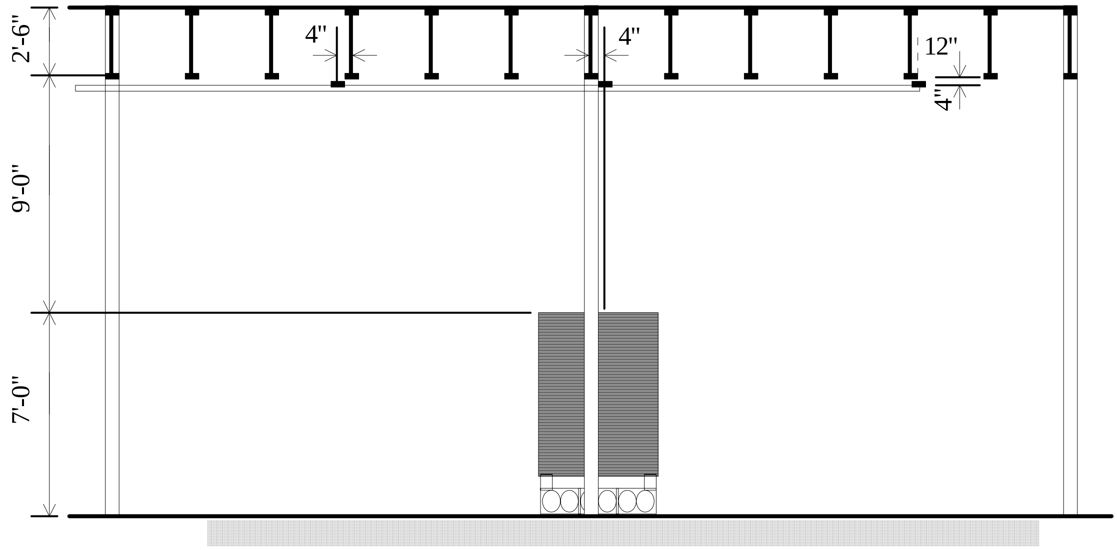

Figure 42.   Elevation View of Test 2

Test 3: This test was developed in response to a concern about the performance of sprinkler heads in a 45 ft. channel with the fire source located between four sprinklers. Blocking panels were removed from the 45 ft. section to increase the channel volume to 300 ft.^{3}. The sprinkler head deflectors were located at 31 in. below the bottom of the deck. Fifteen standard response and 15 quick response sprinklers were deployed.

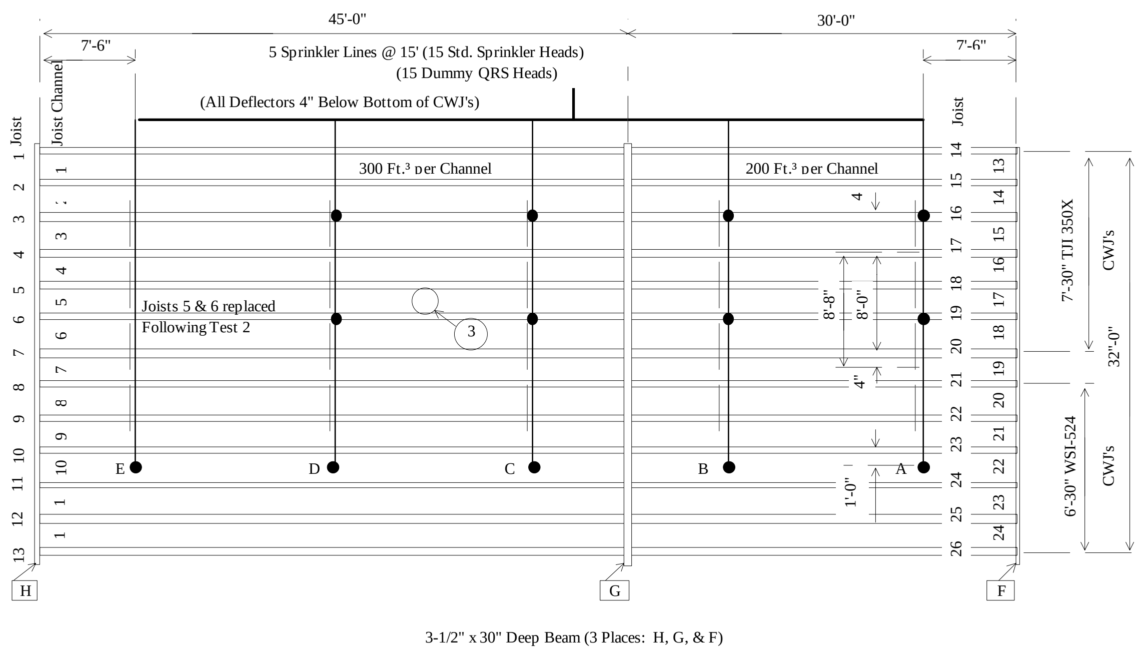

Figure 43.   Plan View of Test 3

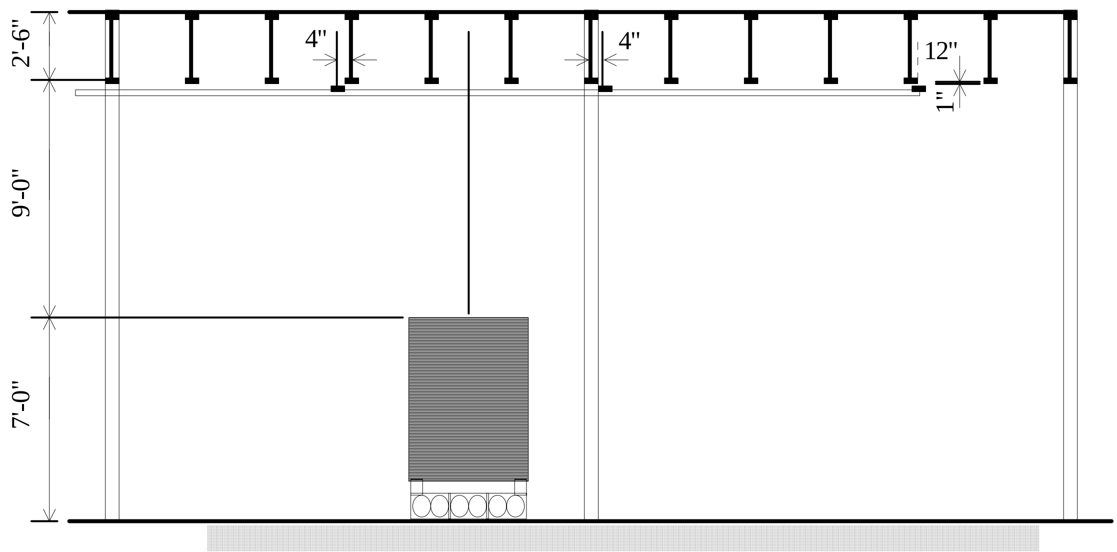

Figure 44. Elevation View of Test 3

Test 4: This test was developed based on a concern that the critical fire location was directly below a firestop. The rest was similar to the general assembly, except that branch lines were located 32 in. to the side of the original locations. The sprinkler head deflectors were located 31 in. below the bottom of the deck. Fifteen inactive quick response and eight active standard response sprinklers were deployed.

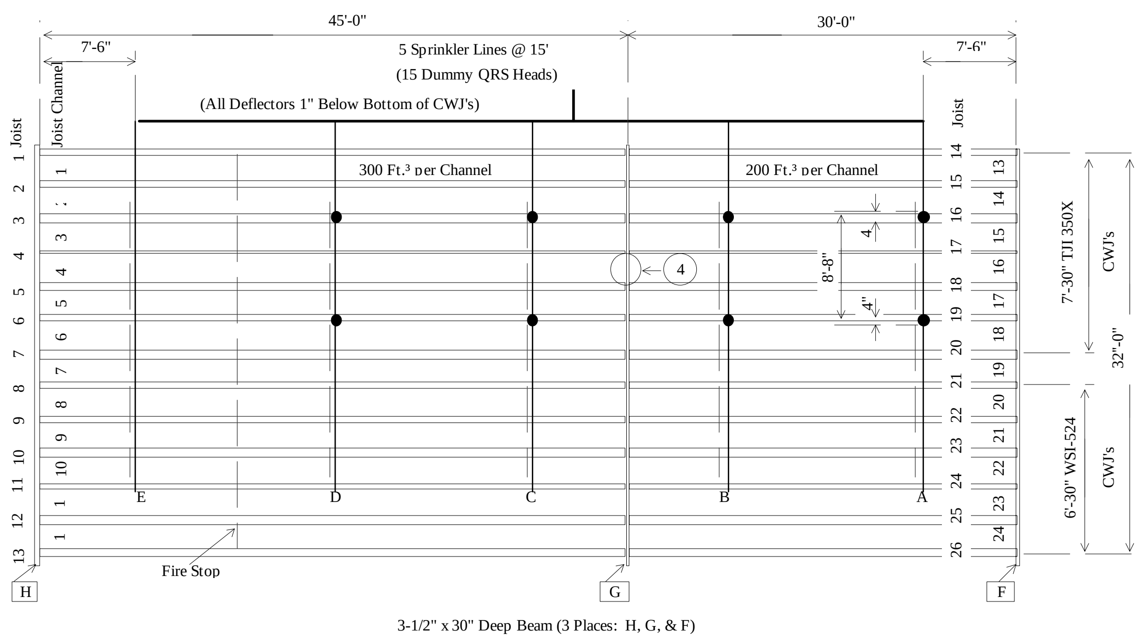

Figure 45. Plan View of Test 4

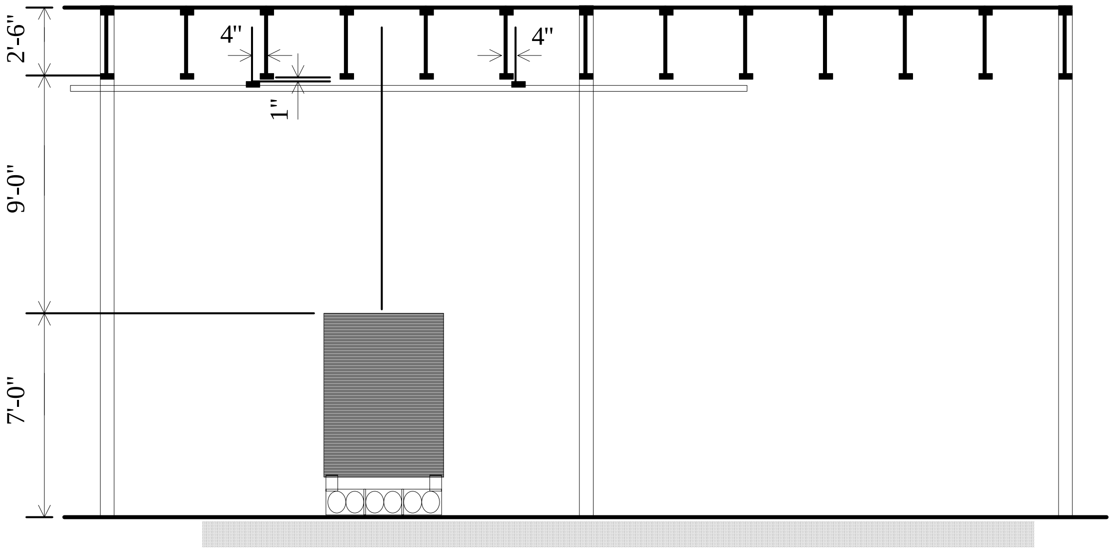

Figure 46. Elevation View of Test 4

Water Supply: The water supply was capable of delivering 1680 gal./min. A static water pressure of 134 lbs./in.^{2} was measured, and a flow density of 0.19 gal./ft.^{2} was maintained throughout each test.

Fire Source: The fire in each test was provided by a wooden crib containing 528 bd. ft. of dimension lumber, as in the previous test. The crib design was similar to that used by Underwriters Laboratories, but contained more than 3-1/2 times the amount of wood.

Report Observations: Temperatures at various locations in the assembly were monitored throughout each test. Visual observations and time to extinguishment were also collected.

Report Summary: Test 1 and 1-1 used the same test configuration as Test 1-2. This configuration was tested three times due to the inability of the sprinkler system to control the crib fire. Test 1 was discounted due to pressure problems noted during the critical first second after activation. Test 1-1 was performed under identical conditions, except deflectors were moved to 31 in. below the deck, instead of 34 in. This scenario also failed to provide adequate fire control.

Test 1-2: This test used only charged quick response sprinkler heads, to determine if an earlier response time would be effective in controlling the fire. The QRS sprinklers activated 45 sec. earlier than standard sprinklers from the previous tests, but the system could not control the crib fire. In Test 1-2, the first QRS sprinkler activated at 2 min., 2 sec., at a temperature of 223° F. This sprinkler was located 4 ft., 4 in. to the left side of crib center. Within 22 sec., three additional sprinklers activated. Despite the activation of four sprinklers, the fire increased in intensity, and the joist channel ignited at 2 min., 51 sec. Two additional QRS sprinklers activated, but did not aid in controlling the fire. Manual assistance was required to extinguish the flames.

Test 2: At 1 min., 1 sec., the charged quick response sprinkler over the fire activated. The charged standard response sprinkler directly over the crib activated at 1 min., 17 sec. This second activation was solely responsible for controlling of the fire. Ten additional charged QRS and four charged standard response sprinklers activated during the course of extinguishment.

The fire was confined to the crib and was allowed to burn for 13 min. before excessive smoke forced manual extinguishment of the fire source. 355 lb. of wood was consumed during the 13 min. fire period. Charring of the joist channel above the fire of 1/16 in. was evident, and some discoloration was also noted. The ultimate load obtained on the most charred joist was 5,866 lb. per reaction. An uncharred control joist was tested to an ultimate load of 7,055 lb. per reaction. This represents a 16.8% reduction in strength.

Test 3: The fire reached the assembly deck at 1 min., 45 sec. into the test. At 2 min., 40 sec., the 45 ft. channel was engulfed in flames. At this point, eight uncharged QRS heads activated. Two charged standard response sprinklers activated at 2 min., 48 sec., soon followed by four additional charged standard response activations. The first four sprinkler heads that activated were those adjacent to the fire. The remaining two standard sprinkler response heads that activated were adjacent to the channel above the fire. The flames within the channel were extinguished by the sprinkler system within 1 min. after activation of the first charged standard response sprinkler head. The fire was confined to the crib. Manual assistance was used to extinguish the fire to prevent excessive smoke buildup in the building.

98 lbs. of wood from the crib burned during the test. Fire damage to the joists due to charring was limited to the 45 ft. section. Two joists experienced charring to a depth of 1/16 in., and there was some discoloration and minor charring on the two joists and the beam. No other damaged was observed. The ultimate strength of the charred joist was 4,748 lbs. of reaction. This compared to an uncharred control joist that had a 5,303 lb. reaction. This represents a 10.5% reduction in strength.

Test 4: The fire source was located directly under the center beam. This forced the fire to burn into both the 45 ft. and 30 ft. channels. At 1 min., 45 sec., flames began to touch the bottom of the joists. At approximately 3 min., six uncharged QRS heads activated. At 3 min., 30 sec., the channels were fully engulfed in flames. Two additional uncharged QRS heads activated at 3 min., 20 sec. Two charged standard response heads activated at 3 min., 40 sec. These heads were the center-most heads, with one head being adjacent to the fire, and one being one line away from the fire. At 3 min., 45 sec., two additional heads activated on the same lines that initially activated. The two heads that activated on the line adjacent to the fire were successful in extinguishing the flames. In total, eight charged standard response sprinklers activated. A line of charged standard response sprinklers directly adjacent to the crib did not activate at all. Two hundred pounds of wood was consumed during the test. Fire damage to the structure was minimal, and remained confined to the channels directly above the fire source. The maximum depth of char noted was 1/8 in. Ultimate strength was measured on three joists, and resulted in strength decreases of 11, 13, and 2.2 percent.

Report Summary:

- When the fire is directly between sprinklers spaced 8 ft., 8 in. apart, the sprinklers are ineffective in controlling the fire. Very little water got to the crib. Any structure type may be threatened under this specific condition. Further evaluation may be desirable, such as the relationship between the sprinkler density of 0.19 gal./min/ft.^{2} and the 850 lb. wood crib.
- In the other three fire scenarios, the sprinklers controlled the crib fire, and the sprinklers protected the structure.
- When the sprinklers controlled the fire, they also protected the structure.

Comments: The last two tests contributed to a change in the NFIPA 13 Standard that allows maximum deflector distance to be 22 in. below the floor or roof deck. This would allow I-joists 22 in. deep to be used under this provision. This has caused concern on the part of those who believe that this deflector distance is too great, given that the performance reliability of this depth joist has not been proven. Part of this controversy arises from the fact that three of the six tests above failed to control the fire.

The tests suggest that when a fire occurs directly between standard or quick response sprinklers spaced 8 ft., 8 in. apart, water distribution patterns do not control the fire. One could conclude that this is due to the joist depth, yet the quick response sprinkler activated at 2 min., 2 sec. in Test 1-2. In the other tests, standard response sprinklers activated between 1 min., 17 sec. and 3 min., 40 sec., and still controlled the fire. This suggests that the crib fire size and its placement between sprinklers creates a water distribution problem that does not contain and suppress the fire. This also suggests that joist depth is independent of this water distribution pattern.

These series of tests provide data needed to understand sprinkler performance at greater deflector distances. These tests also raise a concern over water distribution patterns under one fire load and sprinkler spacing condition. Additional testing would enhance understanding of this anomaly.

5.6 Evaluation of Sprinkler Performance

All sprinkler testing described above was performed under ad hoc conditions. The sprinklers were spaced in accordance with the NFIPA 13 standard and determination was made whether the fire was controlled or extinguished with the particular type of sprinkler, sprinkler orientation, sprinkler water distribution pattern, sprinkler spacing, water flow, and deflector distance, etc. This appears to be one method upon which sprinkler performance is assessed. However, test procedures are not standardized. The development of a consensus standard for sprinkler performance with specific structural member, temperature, and fire control criteria is needed.

Until a standard test protocol and performance criteria are developed, performing additional testing would only provide interesting information. However, there will continue to be the significant possibility that testing like this will be unacceptable to regulatory authorities. This is due to the perception that the tests are biased, or that the testing performed does not meet expectations on how the tests should have been performed. Therefore, it is extremely important for there to be agreement on what constitutes acceptable performance.

Chapter 6: Building Code Requirements

6.1 Model Codes

Three major model building codes in the United States define construction:

- **Uniform Building Code** (UBC) by the International Conference of Building Officials (ICBO)
- **National Building Code** by Building Officials and Code Administrators International (BOCA)
- **Standard Building Code** by Southern Building Code Congress International (SBCCI)

There are also state and local building codes and other jurisdictions that have their own code provisions. Canada has a building code that is different from any of those in the United States. One major difference in the Canadian code that addresses the fire endurance performance of assemblies is an allowance for 45-minute rated assemblies in certain applications.

A mechanism to aid in the use of alternate materials of construction is called an Evaluation Report. Manufacturers of products that do not fall within the specific context of the code can submit test data that will be evaluated by the model code groups for suitability and equivalence with existing code provisions. A report is produced based on their findings, which may allow the product to be used as an alternate method of compliance.

Ultimately, the responsibility for the decision to allow a specific construction type, method or material is determined by the local jurisdiction or building official working on the project.

6.2 Uniform Building Code

While a number of model building codes are produced, and countless other local codes exist, the 1991 edition of the **UBC**, produced by ICBO, has been used here for defining code requirements for structural members.

6.2.1 Type I—Fire Resistive Buildings

Structural elements in Type I Fire Resistive (FR) buildings shall be of steel, iron, concrete or masonry. Under certain conditions within the code, heavy timber members can be used as structural framework or roof framing in Type I buildings, usually for buildings with only one story. In cases where the structural framing is greater than 25 ft. above any floor, every part of the roof frame including the structural frame may be unprotected. In other cases, where every part of the structural steel framing is between 18 and 25 ft. high, the structural members shall be protected by ceiling assemblies of not less than 1-hour FR construction. Generally, Type I construction has no limits on allowable floor area or building heights.

6.2.2 Type II Buildings

The structural elements in Type II-FR buildings shall be of steel, iron, concrete or masonry. The structural elements of Type II-1-hour or Type II-N (N = No fire resistance requirements) buildings shall be of non-combustible materials. Walls and partition systems in both Type II-FR and Type II-1-hour buildings have provisions for the use of fire retardant treated wood stud framing. The allowable grade floor area for Type II-FR ranges from 12,400 ft.^{2} to 59,900 ft.^{2}; for Type II-1-hour from Not Permitted to 27,000 ft.^{2}; and for Type II-Not Protected from Not Permitted to 18,000 ft.^{2}. A maximum height for Type II-FR is 160 ft., for Type II-1-hour is 65 ft., and for Type II is 55 ft. The number of stories is also limited: Type II-FR is limited to 12 stories; Type II-1-hour, to 4 stories; and Type II-N, from Not Allowed to 3 stories.

6.2.3 Type III Buildings

Structural elements in Type III buildings may be of any materials permitted by the code. Type III-1-hour buildings shall be of 1-hour FR construction throughout. The allowable floor area for Type III-1-hour buildings range from Not Permitted to 27,000 ft.^{2}, and for Type III-N from Not Permitted to 18,000 ft.^{2}. The building height for Type III-1-hour buildings is limited to 65 ft., and 55 ft. for Type III-N (not protected) Buildings. The maximum number of stories for Type III-1-hour is four and for Type III-N from Not Permitted to 3 stories. Type III buildings are often referred to as ordinary construction, and are typically built with masonry walls and wood floors and roofs.

6.2.4 Type IV Buildings

Structural elements of Type IV buildings may be of any materials permitted by the code. Type IV construction shall conform to heavy timber construction requirements, except that partitions and members of the structural frame may be of other materials, provided they have a fire resistance of not less than 1-hour. Type IV construction has a range of allowable floor areas from Not Permitted to 27,000 ft.^{2}. The allowable building height is 65 ft. The number of stories ranges from Not Permitted to 4.

6.2.5 Type V Buildings

Type V buildings may be of any material allowed by the code. Type V-1-hour buildings shall be of 1-hour FR construction throughout. The allowable floor area in Type V-1-hour ranges from Not Permitted to 21,000 ft.^{2}. Type V-N ranges from Not Permitted to 12,000 ft.^{2}. The allowable building height for Type V-1-hour is 50 ft. and Type V-N is 40 ft. The number of stories ranges from Not Permitted to three stories for both classifications.

6.3 Allowable Heights and Areas

The code permits area and height increases for all construction types where alternative fire safety features have been provided. These include sprinklers, increased open space, and use of fire walls.

For all construction types under R-3 construction (one- and two family dwellings and lodging houses), the maximum allowable floor area is unlimited, and the maximum allowable number of stories is three.

Sprinkler and standpipe systems are generally detailed in the sections of the code addressing allowable area increases and maximum building height. In general, the areas specified in the code may be tripled in one story buildings, and doubled in buildings with more than one story, if the building is provided with an approved automatic sprinkler system throughout. Also, the building height may be increased by one story. Sprinkler systems are required to be installed in accordance with **UBC** Standards 38-1 and 38-3, which are the 1989 edition of NFPA 13, "Standard for the Installation of Sprinkler Systems," and NFPA 13R , "Standard for the Installation of Sprinkler Systems in Residential Occupancies up to Four Stories in Height," respectively, with minor revisions.

6.4 Comments

As can be seen in this cursory overview of code provisions, each type of construction within the building code allows structural systems to be built without rated fire resistance. This means that in many of those buildings the structural system is not protected by any kind of fire rated membrane or coating. It appears that when such systems are constructed from lightweight building components, there can be concern that fire performance has been compromised in some manner. The code recognizes the increased possibility of greater fire damage in unprotected buildings, and restricts their allowable areas and heights. As greater protection is installed (i.e., 1-hour rated assemblies and sprinklers), greater allowable building sizes and heights can be used. Greater building separation from adjacent buildings allows for increased building areas as well. In general, where fire rated assemblies are used, they are required to have a minimum 1-hour rating. Mixed occupancies necessitate the use of fire resistant assemblies with greater hourly ratings for both walls and floor/ceiling assemblies. The following tables, taken from the **1991 Uniform Building Code**, summarize the fire resistance provisions of the code as it relates to mixed occupancy, allowable areas and heights:

	Types of Construction[1]								
	I	II			III		IV	V	
Occupancy	F.R.	F.R.	1-hour	N	1-hour	N	H.T.	1-hour	N
A-1	Unlimited	29,000	Not Permitted						
A-2-2.1[2]	Unlimited	29,900	13,500	Not Perm.	13,500	Not Perm.	13,500	10,500	Not Perm.
A-3-4[2]	Unlimited	29,900	13,500	9,100	13,500	9,100	13,500	10,500	6,000
B-1-2-3[3]	Unlimited	39,900	18,000	12,000	18,000	12,000	18,000	14,000	8,000
B-4	Unlimited	59,900	27,000	18,000	27,000	18,000	27,000	21,000	12,000
E-1-2-3	Unlimited	45,200	20,200	13,500	20,200	13,500	20,200	15,700	9,100
H-1	15,000	12,400	5,600	3,700	Not Permitted				
H-2[4]	15,000	12,400	5,600	3,700	5,600	3,700	5,600	4,400	2,500
H-3-4-5[4]	Unlimited	24,800	11,200	7,500	11,200	7,500	11,200	8,800	5,100
H-6-7	Unlimited	39,900	18,000	12,000	18,000	12,000	18,000	14,000	8,000
I-1.1-1.2-2	Unlimited	15,100	6,800	Not Perm[8]	6,800	Not Perm.	6,800	5,200	Not Perm.
I-3	Unlimited	15,100	Not Permitted[5]						
M[6]	See Chapter 11								
R-1	Unlimited	29,900	13,500	9,100 [7]	13,500	9,100 [7]	13,500	10,500	6,000[7]
R-3	Unlimited								

1 For multistory buildings, see Section 505(b).
2 For limitations and exceptions, see Section 602.
3 For open parking garages, see Section 709.
4 See Section 903.
5 See Section 1002(b).
6 For agricultural buildings, see also Appendix Chapter 11.
7 For limitations and exceptions, see Section 1202(b).
8 In hospitals and nursing homes, see Section 1002(a) for exception.

N = No requirements for resistance F.R. = Fire Resistance H.T. = Heavy Timber

Table 36. Basic Allowable Floor Area for Buildings One Story in Height (In Square Feet)

	A-1	**A-2**	**A-2.1**	**A-3**	**A-4**	**B-1**	**B-2**	**B-3[1]**	**B-4**	**E**	**H-1**	**H-2**	**H-3**	**H-4-5**	**H-6-7[2]**	**I**	**M[3]**	**R-1**	**R-3**
A-1		N	N	N	N	4	3	3	3	N		4	4	4	4	3	1	1	1
A-2	N		N	N	N	3	1	1	1	N		4	4	4	4	3	1	1	1
A-2.1	N	N		N	N	3	1	1	1	N		4	4	4	4	3	1	1	1
A-3	N	N	N		N	3	N	1	1	N		4	4	4	3	2	1	1	1
A-4	N	N	N	N		3	1	1	1	N		4	4	4	4	3	1	1	1
B-1	4	3	3	3	3		1	1	1	3		2	1	1	1	4	1	3	1
B-2	3	1	1	N	1	1		1	1	1		2	1	1	1	2	1	1	1
B-3[3]	3	1	1	1	1	1	1		1	1		2	1	1	1	3	1	1	1
B-4	3	1	1	1	1	1	1	1		1		2	1	1	1	4	N	1	1
E	N	N	N	N	N	3	1	1	1			4	4	4	3	1	1	1	1
H-1	Not Permitted in Mixed Occupancies. See Chapter 9																		
H-2	4	4	4	4	4	2	2	2	2	4			1	1	2	4	1	4	4
H-3	4	4	4	4	4	1	1	1	1	4		1		1	1	4	1	3	3
H-4-5	4	4	4	4	4	1	1	1	1	4		1	1		1	4	1	3	3
H-6-7[1]	4	4	4	3	4	1	1	1	1	3		2	1	1		4	3	4	4
I	3	3	3	2	3	4	2	3	4	1		4	4	4	4		1	1	1
M[2]	1	1	1	1	1	1	1	1	N	1		1	1	1	3	1		1	1
R-1	1	1	1	1	1	3	1	1	1	1		4	3	3	4	1	1		N
R-3	1	1	1	1	1	1	1	1	1	1		4	3	3	4	1	1	N	

For multistory buildings, see Section 505(b).
1 Open parking garages are excluded, except as provided in Section 702(a)
2 For special provisions on highly toxic materials, see Fire Code.
3 For agricultural buildings, see also Appendix Chapter 11

Table 37. Required Separation in Buildings of Mixed Occupancy (In Hours)

Occupancy	Types of Construction								
	I	II			III		IV	V	
	F.R.	F.R.	1-hour	N	1-hour	N	H.T.	1-hour	N
	Maximum Height in Feet								
	Unlimited	160	65	55	65	55	65	50	40
	Maximum Height in Stories								
A-1	Unlimited	4	Not Permitted						
A-2-2.1	Unlimited	4	2	Not Perm.	2	Not Perm.	2	2	Not Perm.
A-3-4[1]	Unlimited	12	2	1	2	1	2	2	1
B-1-2-3[2]	Unlimited	12	4	2	4	2	4	3	2
B-4	Unlimited	12	4	2	4	2	4	3	2
E[3]	Unlimited	4	2	1	2	1	2	2	1
H-1[4]	1	1	1	1	Not Permitted				
H-2[4]	Unlimited	2	1	1	1	1	1	1	1
H-3-4-5[4]	Unlimited	5	2	1	2	1	2	2	1
H-6-7	3	3	3	2	3	2	3	3	1
I-1.1[5]-1.2	Unlimited	3	1	Not Perm.	1	Not Perm.	1	1	Not Perm.
I-2	Unlimited	3	2	Not Perm.	2	Not Perm.	2	2	Not Perm.
I-3	Unlimited	2	Not Permitted[6]						
M[7]	Unlimited	See Chapter 11							
R-1	Unlimited	12	4	2[8]	4	2[8]	4	3	2[8]
R-3	Unlimited	3	3	3	3	3	3	3	3

Table 38.   Maximum Height of Buildings

To go into greater detail on code requirements for specific occupancies or mixed uses in order to delineate where protected assemblies are used is beyond the scope of this chapter.

6.5 Current Code Environment

In response to concerns about the fire performance of lightweight construction, there have been a variety of recent code changes proposed on both protected and unprotected systems. A description of a few of the proposals that have been under consideration follow:

- In Pointe-Claire, Quebec, a code change that requires 5/8 in. Type C gypsum wallboard on all standard wood joists was passed. Additionally, all other types of floor joists are to be protected on all levels with a minimum of 5/8 in. Type C wallboard, and shall also have all levels equipped with interconnected smoke alarms (including basements, garages and all floor levels), and shall provide fire curtains for every 215 ft.^{2} of concealed space.
- State of Massachusetts' House Bill 820 sought to prohibit the use of trusses in residential construction. Trusses are defined by a legislative committee as I-joists, metal plate connected trusses, and other engineered truss types. This particular bill is currently in committee, and no action has been taken on it at this time.
- The state of New Jersey recently enacted a law that requires an identifying emblem be attached to the front of structures with truss construction. The emblem shall be bright and of reflective color or made of reflective material. The shape of the

emblem shall be an isosceles triangle, and the size shall be 6 in. high by 12 in. long. An 'F' inside the triangle will specify a floor truss, 'R' a roof truss, and 'FR' both construction types. Detached one- and two-family dwellings that are not part of a planned real estate development are exempt from these provisions. Individual structures and dwellings that are part of a planned real estate development shall not be required to have an identifying emblem if there is an emblem affixed to the development. The governing body of the municipality may require, by ordinance, that emblems be affixed on any structure using truss construction. This law left truss construction undefined, so it applies to all "trusses". The bill was developed in response to the Hackensack, New Jersey fire previously described.

- The city of Rockford, Illinois, was considering an ordinance that would require all structured elements used in floor or roof systems to have fire endurance performance equivalent to solid-sawn joists. As of December, 1991, no action had been taken.
- A provision exists in Palatine, Illinois, that requires 1/2 in. Type X gypsum wallboard to be applied on all lightweight components used in floor or roof assemblies. This provision does not apply to solid-sawn joists.
- In British Columbia, a proposed change to the provincial building code would require nails or staples to be installed on the metal plates of trusses, to prevent them from falling out in a fire. This concept was rejected by the Code Committee in December, 1991.
- A code change was proposed at the 1991 BOCA code change hearings that would have excluded metal plate connected roof trusses from buildings in Use Group R (residential construction), unless the building was equipped throughout with an automatic sprinkler system in accordance with the appropriate BOCA sprinkler sections. This code change did not pass.
- The city of Glen Cove, New York ratified an amendment to its building code that stated that all new or modified construction utilizing prefabricated support structures consisting of wood truss members with steel plate connectors in floor/ceiling assemblies shall require the formal notification of the city building department administrator. Wherever wood truss members with steel plate connectors are used, buildings shall be equipped with a sprinkler system (which sprays both up and down) in all voids and attic spaces in accordance with NFPA 13. The status of enforcement of this provision is unknown at this time.
- A proposed code change in Laval, Quebec, would require that buildings having roofs or floors constructed out of metal web trusses, I-joists, or other similar construction systems be sprinklered in conformance with NFiPA 13, 13D, or 13R.
- The city of Long Grove, Illinois, among others, requires that all residential construction be built with sprinkler system protection.

When the evidence submitted by the code change proponent is reviewed, it is easy to conclude that many of these proposals are in response to the articles reviewed in **Chapter 2** of this report. Much of the substantiating language and thoughts appear to be taken directly from some of the fire service articles reviewed. An example of this is seen in the proposed 1991 BOCA code change submission, B 253-91:

> *Lightweight wood trusses from the construction perspective all share the same basic advantages, but there are disastrous disadvantages for fire fighters. By engineering calculations and practical fire fighting experience, lightweight trussed rafters may be expected to collapse after approximately ten minutes in a fully involved fire. During a fire it takes time for the fire to deteriorate the rafters (in this type of construction) to a point where they give way. This gives fire fighters enough time to perform ventilation on a roof that is strong enough to hold several fire fighters at one time. The lightweight wood truss is a fast burner. This is compounded by the problem that if one part of the truss fails, the entire truss fails. Whereas, the failure of one rafter will not cause the failure of any other element.*
>
> *Lightweight wood truss construction involves large interconnected areas in which fire can be hidden and explosive or back draft heated gases can accumulate. It is possible to have a serious fire in a roof void with little or no smoke visible in a building. If a fire enters any attic or other concealed space it will spread rapidly and involve the entire area. A solid wood framing member will cause fire blocking for a period of time, there is no such fire blocking with the open construction of a lightweight wood truss.*
>
> *Lightweight wood trusses themselves are often manufactured with the use of sheet metal surface fasteners. These fasteners only connect the outer one-half inch wood. Truss design as an architectural design can be defended but the use of sheet metal fasteners cannot. This device is a dangerous structural connection.*[1]

Another factor used to justify code changes is often performance experience. Actual and perceived poor performance causes local code authorities to request that changes be made to the local building code.

6.6 Evaluation of Building Code Requirements

As noted, building codes allow unprotected construction to be used in a variety of occupancy types with a variety of building heights and areas. Thus, lightweight component construction elements can be used in many buildings. Use of fire rated assemblies results from code requirements. The broadest category is 1-hour rated assemblies. As there is no requirement in the code for a fire resistive rating for

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[1] Prepared by Arnold R. Hamilton, East Lake Fire Department, East Lake, Ohio as a BOCA code change proposal. The proposal is printed verbatim.

unprotected assemblies, very little standardized testing has been performed. None has been performed for model code compliance.

Incorporation of sprinkler systems generally allows building size to be increased in both unprotected and protected (i.e., fire-resistance rated) construction.

The 1990 edition of the BOCA **National Building Code** recognizes a life safety benefit of sprinklers by requiring installation in R-1 (hotels, motels, boarding houses, etc.); R-2 (apartments, dormitories, etc.); A-1, A-2, and A-3 (assembly); high hazard; institutional, mercantile S-1 (storage) and F-1 (factory) buildings, albeit with exceptions. The BOCA code routinely allows the reduction of area-separation fire-rated-assembly duration requirements when sprinklers are used. For example, a 2-hour fire separation assembly used in multi-family construction can be reduced to a 1-hour fire separation assembly with an approved automatic sprinkler system. In certain applications, a recent BOCA code change allows for the use of a 30-minute rated assembly with sprinklers in lieu of a 1-hour rated assembly with sprinklers or 2-hour fire separation assembly requirement in multi-family building dwellings, provided that sprinklers are installed in all closets located against tenant separation walls and in all bathrooms. This code change indicates a BOCA recognition that sprinklers have a proven performance history of containing and suppressing fires, ultimately saving lives and property. It also recognizes that the majority of fires begin inside an area that is compartmentalized.

There have been attempts to restrict the use of lightweight components through the code change process. Generally this has been done at the local level, but proposals are being made at the model code level as well. It is often easier for local codes to be changed because of a lack of a formalized code change process. A concern that must be considered is that, in general, those involved with local code changes are not as knowledgeable about the technical details of the variety of products that are available for construction. Their decisions are also heavily influenced by the current published literature on the topic. Therefore, it is imperative that facts be presented accurately on technical topics.

At the model code level, a code change proponent must provide detailed substantiation and reasoning for a code change to be adopted. The code change must then face a consensus vote of the entire voting membership of the model code body (typically limited to building officials only). This process is meant to screen and evaluate code change proposals, and adopt those that are technically supportable. The BOCA code change described above (B253-91) is an example of a change that was not adopted because of the absence of adequate substantiating evidence.

Nonetheless, the code change trend is a concern, given the number of code changes being submitted with little substantiating data. This is particularly true for changes proposed at the local level, where there is often less of a need for substantiating data, and no formalized consensus-based code change process in place to require technical rigor. This can lead to costly, ineffective, and technically unsound public policy decisions.

Chapter 7: Discussion

7.1 Lightweight Building Component Fire Performance Issues

The following discussion is based on the data found in the preceding chapters. Because the breadth of this subject is great, it is difficult to reduce it to a few simple points. Statistics, test data, and model code considerations are discussed first, followed by a discussion of the concerns brought forward by the fire service.

7.2 Fire Loss Statistics

As was seen in **Chapter 3**, the majority of fires in the United States occur in residences (one- and two-family dwellings and apartments). The majority of those fires begin in living areas (e.g., kitchen, living room, bedroom, etc.) that are typically compartmentalized. This means that structural support members for walls, floors and ceilings are sheathed, and therefore protected. For walls and ceilings, this protection is typically provided by gypsum wallboard. Floors are generally constructed of plywood, concrete, or gypcrete. Given the statistics, the focus on fire performance of lightweight engineered building components ought to be directed more toward the various fire performance aspects of protected assemblies, than other areas of potential study.

Since most fires begin in compartmentalized living spaces, the addition of sprinklers would prevent losses of life and property in many of the fires that occur, and reduce the risk to the fire service. This confirms what has been known for a long time. There are two key fire safety measures that will reduce loss of life in fires: the use of smoke detectors and the use of sprinkler systems.

Finally, a risk assessment should be performed that considers the various causes of firefighter fatalities. This risk assessment can then be used to develop firefighting tactics that will help reduce the risk of fatality. Ideally, the risk of death on the fireground should be reduced to as close to zero as is reasonable possible.

7.3 Summary of Testing

In **Chapter 4**, available testing performed on engineered components has been compiled and reviewed. A discussion of each section of **Chapter 4** follows:

7.3.1 Unsheathed Assemblies (Chapter 4-1)

Testing described in **Section 4-1.1** concluded that each of the lightweight building component members tested resulted in early failure.[1] However, "early failure" is not yet

[1] Mittendorf, J., "Lightweight Construction Tests Opens Fire Service Eyes to Special Hazards," Western Fire Journal, January, 1982.

well defined, and the test procedures were not standardized, so that comparative performance of the structural elements cannot be accurately assessed.

The Illinois Fire Service Institute tests provide some indications of warning signals that may be available for each of the structural components tested.[2] Those tests noted that:

- 2 x 10s gave ample warning by the sagging of the structural system.
- MPC wood trusses sagged, giving a definite indication of structural problems.
- Metal web wood trusses sagged early, giving an indication of structural problems.
- Wooden I-joists did not sag or produce warning noises to indicate there were structural problems.
- Pin-end connected steel webbed wood trusses also failed without sagging or providing any warning.

However, since this testing was also performed without standardized test procedures, only a qualitative assessment of potential differences between components can be made.

When standardized tests at full design load are studied (See Table 22, page 108), it can be seen that deflection at failure is significant for the truss assembly and the two steel C-joist assemblies. The deflection at failure is 11-1/2 inches for the truss assembly, and 7 and 10 inches for the steel C-joist assemblies. The 2 x 10 deflection performance was in the range of 2.7 to 4 inches at failure. Given this, it could be concluded that the failure warning signals for trusses and steel C-joists may be more significant, in terms of deflection, than typical joist construction. It was also noted in the testing that as the loading decreased, the associated deflection near failure decreased as well. This may indicate that under typical room loading conditions (which are typically far less than design load), the warning that exists through deflection performance may not be as noticeable. This has significant ramifications on the fireground.

Room fire tests are interesting as they provide data intended to represent performance under more realistic fire load conditions. This testing also indicated that the deflection performance of both steel joists and 2 x 8 wood joists is significant near failure. However, in all cases, the load applied to the system was near the maximum allowable design load, which could overstate the deflection that would be seen at a typical fire scene.

None of the standardized tests record indications or warning signals that could be expected prior to collapse, because information of this nature is not typically noted in these test reports. If this information is desired, specific test procedures should be developed to detail the warning signals available prior to failure of the tested assembly.

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[2] Straseske, J. and Weber, C., "Testing Floor Systems," Fire Command, June, 1988.

A time/temperature curve was developed to represent typical room fire-load conditions, which are defined in the test report.[3] This new curve caused failure to occur much more quickly than in the room burns it represented. At this time, it is difficult to evaluate the usefulness of the new time/temperature curve. More testing should be performed, and the curve should be calibrated to actual room fire test results, so it can be assured that it accurately represents a realistic room fire.

Finally, the available test data that allows for direct comparison between assemblies can be reduced to eight tests (see Table 22, page 108). These tests indicate that in unsheathed assemblies, wood joists have greater fire endurance than steel C-joists. The data also indicate that MPC trusses have fire endurance times that fall close to the range of performance for 2 x 10 joists. However, the MPC truss assembly tested did not have a splice plate located in the bottom chord of the truss. It is expected that this would reduce the time to failure, although it is unknown by what amount.

There are currently no fire endurance performance criteria available or that must be met for unsheathed assemblies based on this literature review. It is expected that this lack of performance criteria, and the fact that there has never been a requirement or proposal to test unsheathed assemblies, are the reasons for the small amount of standardized test data available on unsheathed fire endurance assemblies.

7.3.2 Single Membrane Protected Assemblies (Chapter 4-2)

The testing of a single gypsum wallboard membrane directly attached to structural elements yielded the following results:

- Assemblies with 1/2-inch fire rated Type X gypsum wallboard applied directly to 2 x 10 joists typically have a 45 minute assembly rating.
- Assemblies with 5/8-inch fire rated Type C gypsum wallboard applied directly to wooden I-joists, MPC trusses or pin end connected steel web trusses have a 45-minute assembly rating.

In each case, the assembly's performance duration was determined by structural failure.

In all tests performed with a single membrane applied directly to the structural element, there was deflection prior to collapse, ranging from very little to quite noticeable. This deflection ranged from 1.03 to 12.9 inches for trusses and I-joists, and 1.85 to 13.0 inches for 2 x 10 joists. Unfortunately, collapse warning signals (e.g., rate of deflection) were not recorded as part of the test procedure. Therefore, it is difficult to determine the types of collapse warning signals that may exist prior to collapse, other than the system deflection, which is only valuable when the deflection magnitude is significant. In the

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[3] Fang, J.B., Fire Endurance Tests of Selected Residential Floor Constructions, NBSIR 82-2488, U.S. Department of Housing & Urban Development, April 1982. See summary in **Sections 4-1.14** and **4-2.10** of this report.

cases where the deflection is less than 2 inches, the value of deflection as a warning signal is not as great.

If a fire begins in a properly constructed compartment which has 5/8-inch thick fire-rated gypsum wallboard on horizontal lightweight engineered components, the rating for this compartment will typically be 45 minutes. For solid sawn joist construction, the equivalent rating is achieved with a 1/2-inch thickness of fire-rated gypsum wallboard.

Any compartment with 1/2-inch regular gypsum wallboard attached to the structural elements should have fire resistance performance of at least 15 minutes, since the gypsum wallboard membrane provides a 15 minute membrane rating.[4,5] The fire endurance performance of the structural members will add to the 15 minute membrane performance. Thus, most residences will have protection slightly greater than 15 minutes, should a fire start in a living area that has wallboard sheathing. This suggests that the fire performance of any unsheathed system can be increased to at least 15 minutes by attaching a single layer of 1/2-in. regular gypsum wallboard directly to the unsheathed structural system. This concept is supported by the test data found in this chapter.

The most "realistic" data found in the literature were three protected tests performed by the National Bureau of Standards using actual room fire conditions.[6] Additional testing of this type would probably be very valuable for the fire safety community in terms of developing the warning signals that occur prior to collapse, collapse mechanisms, failure modes and the deflection performance of the various assemblies under more realistic fire conditions. This type of testing has excellent potential for being very valuable, if test methods are developed to specifically yield results that provide information that can be used to improve tactical fireground approaches.

7.3.3 Connections (Chapter 4-3)

Firefighters are concerned with the performance of different types of connections in fire conditions. The literature revealed six test reports that were concerned specifically with the fire endurance performance of connections. This testing was not standardized, and presently, there are no standardized test procedures or performance requirements for evaluating only connections placed under fire test conditions. Since engineering design does not take into account the fire performance of connections, additional data must be developed to draw any relevant conclusions on their fire performance. Predicting

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[4] 1991 U.B.C. Standards, "Method for Calculating Fire Resistance of Steel, Concrete and Wood Construction," U.B.C. Standard 43-9, Table No. 43-9-W-A, Pg. 1518.

[5] 1988 Standard Building Code, "Calculating Fire Resistance," Chapter 31, Table 3106.2A, Pg. 469.

[6] Fang, J.B., Fire Performance of Selected Residential Floor Constructions Under Room Burnout Conditions, NBSIR 80-2134, December, 1980. See **Sections 4-1.13** and **4-2.9** of this report.

performance using these small sets of data would not be recommended, as comparative results would be questionable due to the lack of statistical significance.

Currently, connections are always evaluated as an integral part of the fire endurance assembly being tested.

The fire endurance performance of connections is an area where additional data would be useful to better evaluate and understand performance characteristics.

7.3.3.1. Truss Plate Connectors

Testing performed on metal plate connectors (MPCs) generally indicates performance of less than 10 minutes. Testing currently being conducted at the United States Department of Agriculture Forest Products Laboratory holds great promise for adding to the MPC fire performance database, and for creation of a model that will predict the performance of a single MPC truss element under fire load conditions.[7] From this, the capability of predicting the performance of an entire truss assembly is expected to follow.

7.3.4 Operation Breakthrough Assemblies (Chapter 4-4)

The objective of Operation Breakthrough was to test the fire endurance performance of many systems that could potentially be used in the manufactured housing environment. Therefore, there was little standardized testing was done for purposes of direct comparison, but rather, a variety of tests were performed to determine the performance of specific easily manufactured assemblies.

Operation Breakthrough did yield some information about systems using double layers of 1/2-inch fire-rated gypsum. These systems showed performance of a joist-rafter assembly and a steel C-joist assembly that go well beyond a 60 minute rating. This finding is typical for double layer 1/2-inch Type X gypsum wallboard fire endurance assemblies. When existing data (e.g., industry test data not included in this report) on two-layer 1/2-inch fire rated gypsum assemblies are combined with these data, it becomes apparent that a two-layer 1/2-inch Type X gypsum wallboard system generally provides at least one hour of fire resistance performance when attached to almost any structural floor framing system. However, this performance will be dependent on secure attachment of the two layers of wallboard to the framing members.

The Operation Breakthrough data can only be used for general observations, and to gain knowledge regarding the performance of the specific types of assemblies under the fire endurance conditions described in the test report.

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[7] White, R.H., Cramer, S.M., and Wolf, R.W., National Forest Products Association Committee on Research and Evaluation Report, April 4, 1991.

7.3.5 Coating Performance (Chapter 4-6)

The literature search produced a very small amount of information regarding the tested performance of coatings on MPC connections and steel bar joists. It is well known, however, that there is a body of data available, and that model building codes have developed calculation procedures for insulating steel beams, columns and joists from fire through the use of coatings. Concrete is also used as a protective coating.

From the limited testing available, coatings enhance the performance of connection and structural systems under fire load conditions. Additional testing will be necessary to determine the degree of performance improvement and how coatings can be economically employed to improve lightweight building component fire endurance systems.

7.3.6 Sprinkler Performance (Chapter 5)

The literature reveals that there is no standardized test procedure available to evaluate the performance of sprinklers attached to a given structural framing system, or for sprinklers employed within a concealed space. The available testing provides only a small base of information upon which to evaluate the performance of sprinkler systems used with wooden I-joists and MPC trusses. Unfortunately, no pass/fail criteria have been defined for these types of tests; therefore, no measure of acceptable performance is available. Without such criteria, any testing performed is subject to criticism, and may be considered unacceptable. A consensus standard and associated performance acceptance criteria for the testing of structural elements that support sprinkler systems may be needed.

The I-joist testing demonstrated that there may be a fire load size and placement that current sprinkler technology does not adequately contain or extinguish. This condition is a cause for concern, and should be more thoroughly evaluated, since there may be certain field applications that are at risk.

The quoted tests performed by the City of Fort Worth were used to make a local code change (See **Section 5.3** for test details). The sprinkler layout and positioning are not supported or rejected by this project's Technical Advisory Committee because they may not result in adequate sprinkler protection for the building or the structural support system.

Test methods and evaluation criteria need development, and testing will need to be performed in the future to address sprinkler performance when used with lightweight building components.

An additional concern of the fire service is the manner in which sprinklers can be installed. When NFiPA 13[8] is followed, it is presumed the building is sprinklered throughout. NFiPA 13R[9] and 13D[10] allow for there to be some areas of the building that are not sprinklered. For example, NFiPA 13R and 13D allows sprinklers to be omitted from the following areas:

- Bathrooms not exceeding 55 sq.ft., with non-combustible plumbing fixtures.
- Small clothes closets where the least dimension is 3 ft., the area doesn't exceed 20 sq.ft., (24 sq.ft. in 13D) and the walls and ceiling are surfaced with non-combustible or limited-combustible materials.
- Open attached: porches, balconies, corridors, and stairs.
- Attics, penthouses, equipment rooms, crawl spaces, floor/ceiling spaces, elevator shafts, and other concealed spaces that are not used for intended for living purposes or storage.
- Sprinklers may be omitted from entrance foyers which are not the only means of egress (13D only).

Should a fire begin in one of these areas, it is uncertain how the remaining sprinklers will function in controlling the fire, if it is controlled at all.[11][12]

7.3.7 Summary of Test Data

To gain an appreciation for where test data is and is not available, the following tables have been prepared:

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8 NFPA 13, **Installation of Sprinkler Systems**, 1987 ed.

9 NFPA 13R, **Standard for the Installation of Sprinkler Systems in Residential Occupancies up to Four Stories in Height**, 1989 ed.

10 NFPA 13D, **Sprinkler Systems - One- and Two-Family Dwellings**, 1984 ed.

11 NFPA 13R, ***loc. cit.***

12 NFPA 13D, *loc. cit.*

Description[1]	**Full Design Load**[2]	**Restricted Load**[3]	**Small-Scale**[4]	**Ad-Hoc**	**Room Burn**	**Full Bldg.**	**Other (e.g., ISO 834)**
Wood Joists	9*	5*	1*	1*	2[1]	N/A	3 Room T/T
MPC Trusses	1*	*	1*	3*	N/A	N/A	N/A
MPCMW Trusses	N/A	N/A	1*	1*	N/A	N/A	N/A
I-Joists	N/A	N/A	1*	2*	N/A	N/A	N/A
PECMW Trusses	N/A	N/A	N/A	2	N/A	N/A	N/A
Steel Bar Joists	N/A*	1*	N/A	1*	N/A	N/A	N/A
Steel Joists	3*	3*	N/A	N/A	2	N/A	1 Room T/T
Heavy Timber	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Glulam	1	N/A	N/A	N/A	N/A	N/A	N/A
Panelized	N/A	N/A	N/A	1*	N/A	N/A	N/A
Sandwich Panel	3	1	N/A	N/A*	N/A	N/A	N/A
Steel Beams	1*	N/A*	N/A*	N/A*	N/A*	N/A*	N/A
Truss Plate Con. Joists	N/A	4*	N/A	N/A*	N/A	N/A	1 ISO 834

* More tests may be available from proprietary sources.

1 For report details, see **Chapter 4-1: Fire Endurance Performance of Unsheathed Assemblies**.

2 Follows the standard ASTM E119 test method using time/temperature curve and the maximum allowable design load.

3 Follows the ASTM E119 standard test method using the time/temperature curve and a less-than-maximum allowable design load with actual load applied recorded.

4 Uses the ASTM E119 time/temperature curve in a small size furnace at typically much less than full design load conditions. In some cases, with no load at all.

N/A No tests available through the literature search process. May be available from proprietary sources.

Table 39. Number of Tests Performed on Unsheathed Assemblies from the Test Reports Available.

Description[1,5]	Full Design Load[2]	Restricted Load[3]	Small-Scale[4]	Ad-Hoc	Room Burn	Full Bldg.	Other (e.g., ISO 834)
Wood Joists	7 [6]	*	N/A	N/A	1	N/A	2 Room T/T
MPC trusses	3*[6]	*	N/A	N/A	1	N/A	N/A
MPCMW Trusses	*	N/A	N/A	N/A	N/A	N/A	N/A
I-Joists	1*[6]	N/A	*	N/A	N/A	N/A	ISO 834[7]
PECMW Trusses	Few*	*	*	N/A	N/A	N/A	N/A
Steel Bar Joists	*[6]	*	*	N/A	N/A	N/A	N/A
Steel Joists	5*	N/A	*	N/A	1	N/A	N/A
Heavy Timber	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Glulam	*	N/A	N/A	N/A	N/A	N/A	N/A
Panelized	*	N/A	N/A	N/A	N/A	N/A	N/A
Sandwich Panel	*	N/A	N/A	N/A	N/A	N/A	N/A

* More tests may be available from proprietary sources.
1 For report details, see **Chapter 4-2: Fire Endurance Performance of Single Membrane Protected Assemblies**.
2 Follows the standard ASTM E119 test method using time/temperature curve and the maximum allowable design load.
3 Follows the ASTM E119 standard test method using the time/temperature curve and a less-than-maximum allowable design load with actual load applied recorded.
4 Uses the ASTM E119 time/temperature curve in a small size furnace at typically much less than full design load conditions. In some cases, with no load at all.
5 Some cells have information in them that is not discussed in this report.
6 Many more tests are available with a variety of protection systems from proprietary sources.
7 This is an APA test performed at UL.
N/A no tests available through the public literature search process. May be available through proprietary sources.

Table 40. Number of Tests Performed on Protected Assemblies from the Test Reports Available.

Description[1]	**Full Design Load**[2]	**Restricted Load**[3]	**Small-Scale**[4]	**Ad-Hoc**	**Other (e.g., ISO 834)**
Steel Connection Systems	N/A	N/A	N/A	N/A	N/A
Truss Plates	2	1	N/A	4*	N/A
Bolts	1	N/A	N/A	3*	N/A
Nails	1	N/A	N/A	2*	N/A
Split Rings	1	N/A	N/A	1*	N/A
Lag Screws	N/A	N/A	N/A	N/A	N/A
Steel Pins	N/A	N/A	N/A	N/A	N/A
Plywood Gusset	1	N/A	N/A	2*	N/A
Steel Gusset	N/A	N/A	N/A	3*	N/A

* More tests may be available from proprietary sources.

1 For report details, see **Chapter 4-3: Fire Endurance Performance of Connections**.

2 Follows the standard ASTM E119 test method using time/temperature curve and the maximum allowable design load.

3 Follows the ASTM E119 standard test method using the time/temperature curve and a less-than-maximum allowable design load with actual load applied recorded.

4 Uses the ASTM E119 time/temperature curve in a small size furnace at typically much less than full design load conditions. In some cases, with no load at all.

N/A no tests available through the public literature search process. May be available through proprietary sources.

Table 41. Number of Tests Performed on Connections from the Test Reports Available.

Description[1]	**Ad-Hoc**	**Room Burn**	**Full Bldg.**	**Other (e.g., ISO 834)**
Wood Joists	*	*	N/A	N/A
MPC Trusses	12	N/A	N/A	N/A
MPCMW Trusses	N/A	N/A	N/A	N/A
I-Joists	9	N/A	N/A	N/A
PECMW Trusses	N/A	N/A	N/A	N/A
Steel Bar Joists	*	*	N/A	N/A
Steel Joists	N/A	N/A	N/A	N/A
Heavy Timber	N/A	N/A	N/A	N/A
Glulam	N/A	N/A	N/A	N/A
Panelized	N/A	N/A	N/A	N/A
Sandwich Panel	N/A	N/A	N/A	N/A

* Many tests have been done with these structural members, but are not included in this report. No test standard is available; therefore, these are all considered to be ad hoc tests.

1 For report details, see **Chapter 5: Sprinkler Testing**. Some cells have information in them that is not discussed in this report.

N/A no tests available through the public literature search process. May be available through proprietary sources.

Table 42. Number of Tests Performed on Sprinklers from the Test Reports Available.

As can be seen from the summaries, the majority of data available comes from tests performed on protected assemblies. The data presented in this report are only a small fraction of the data available on protected systems. This is logical, since building codes mandate protection of assemblies for a given period of time using ASTM E119 as the standard method of acceptance. Manufacturers wanting to have their product used must comply with code requirements, resulting in an abundance of code compliance testing.

There is relatively little test information on the fire performance of connections and unsheathed assemblies. This is due to the fact that there are no specific code-mandated performance requirements in these areas. Therefore, testing has only been done for evaluation of a specific problem or for general scientific purposes.

Unsheathed tests may provide very useful information for fire service personnel. Results would give a sense for the modes of failure, warning signals prior to collapse and deflection performance of lightweight building components, and provide a basis upon which to build a tactical response. However, the usefulness of unsheathed test information detailed in this report is limited primarily to the data generated from standardized test procedures (See Table 22, page 1088). Each of these tests was performed for a specific purpose, and many of the tests were performed years ago. If a test program is to be developed for unsheathed assemblies, it would be best to perform all testing at a single test facility, under identical test protocols.

The connection test data generated from standardized test procedures included in this report do *not* allow for the evaluation of the performance of both connectors in assemblies and connections within a building structural element in a fire. By testing single connections, one can learn about relative performance and begin to estimate the impact of the connection on the fire performance of the structural member, and ultimately, on the fire performance when the connector is part of an assembly.

Many sprinkler tests have been performed with wood joists and steel bar joists as the structural member supporting the sprinkler system. The testing conducted, however, was intended to examine sprinkler distribution patterns and their ability to control or suppress the fire. Therefore, there is very little information on the fire performance of the structural element supporting the sprinkler system under fire conditions. Currently, the attachment of sprinkler systems to structural members is defined by NFiPA 13, and the physical connection strengths required can be calculated through the use of traditional engineering formulas. The base of knowledge on the fire performance of structural members with attached sprinklers should be more fully developed, as sprinklers will be a more prevalent fire suppression method and life safety tool in the future.

7.4 Model Code Considerations

As seen in **Chapter 6: Building Code Requirements**, the provisions of the model building codes allow for the use of unprotected assemblies. The only constraint on the use of these assemblies in building construction is the type of building and its allowable area and height. Allowable building areas and heights increase with increased protection, for example, through the use of fire endurance rated assemblies for fire compartmentation and sprinklers.

The model building codes recognize the fire safety benefit of sprinklers. ICBO requires sprinklers in apartments, congregate residences and hotels three or more stories in height with the additional provision for number of occupants or dwelling units. The BOCA building code requires sprinklers in residential occupancies such as hotels, motels, boarding houses, apartments and dormitories. A recent BOCA code change allows for the use of a 30-minute rated assembly with sprinklers in certain applications, which is deemed to provide equal to, or more, protection than a typical 1-hour assembly. This is a reduction from the previous one-hour fire-rated assembly with sprinklers or two-hour rated assembly requirements. This code change acknowledges that sprinklers have a proven history of performance in containing and suppressing fires.

Finally, codes are beginning to be changed to restrict the use of lightweight building components due to concern over their fire performance characteristics. There is a concern that code changes may be made without the use of detailed substantiating test or other relevant data. This is particularly true for changes being made in codes at the local level, where there is no formalized code change process requiring the use of substantiated technical data. Code changes should only be made where there is a solid technical basis for the change. To make a code change on any other basis will lead to costly, ineffective and technically unsound public policy decisions.

7.5 Review of Firefighting Concerns

The literature often contains emotional language (e.g., "firefighter's enemy," "killer connector," etc.) to make a point about firefighter safety. This emotion is used to motivate all firefighters to become aware of potential dangers, and to avoid a complacency in learning about the hazards of burning construction and safe firegighting precautions and methods. This approach is useful from a safety awareness standpoint, but must be used with discretion because it can easily be misinterpreted. Understanding all of the technical aspects of this issue is crucial to making valid decision on the fire performance of engineered components.

7.5.1 Product Design and Effect of Mass

It is clear that engineered products are designed to maximize strength and minimize the amount of material going into the product. This minimizes mass. Therefore, to the extent that this mass reduction can create a fire endurance performance problem, engineered products are effected. Many times, however, design for serviceability (e.g., deflection performance—common for component floor assemblies where spans exceed 16 to 18 ft.) is a more significant engineering criterion than design for strength (for all construction elements). This means that the mass of the engineered component will be greater than is actually needed to carry design loads. This may provide reserve strength, which will benefit the fire performance of members. It is also common for structures to be designed to carry greater loads than those that actually occur in the field. This also provides reserve strength that may be beneficial under fire conditions.

Testing of unsheathed wood 2 x 10 joist systems and metal plate connected (MPC) trusses suggests that the mass effect is at best a minor factor in fire endurance performance *when comparing solely these two member types*. For example, tests outlined in **Chapter 4-1—**Mutual Corporation Design FC-250 in **Section 4-1.8**, the Forest Products Laboratory tests in **Section 4-1.7**[13], Factory Mutual Design FC-209 in **Section 4-1.4**, and FC-212 in **Section 4-1.5—**show that unsheathed 12 in. deep parallel chord trusses have a fire endurance performance time of 10 min., 12 sec., and unsheathed 2 x 10 joists have an endurance time of 6 min., 30 sec.[14]; 13 min., 34 sec.; and 12 min., 6 sec.; respectively. The fact that these tests were run under identical fire (ASTM E119), spacing, sheathing, and full design load conditions suggests that the mass difference of the trusses was a minor factor in their fire endurance performance. This is probably due to the fact that the char rate of wood is fairly consistent among wood species, and that in the case of the truss, the ability to carry the applied load will remain until the char layer is deep enough to cause the remaining uncharred wood to fail in bottom chord tension. The same concept is true for a 2 x 10 joist. The dimensions of the bottom chord of the truss are 3.5 in. by 1.5 in., and for the 2 x 10 are 1.5 in. by 9.25 in. In both cases the 1.5 in. dimension is the critical dimension for fire endurance performance. Once the char reduces the 1.5 in. dimension enough the member will fail; hence, similar failure times for these two products.

Mass does become a consideration when comparing 2 x 10 joists with 6 x 20 glulam beams, for example. A 6 x 20 beam will perform much better under fire conditions. It will also be a consideration with engineered components, should *the effective* fire resisting mass decrease. The actual mass effect on fire performance of the engineered components under study needs to be documented more thoroughly, however, because there is not a solid base of data available to accurately evaluate mass effects upon the fire endurance performance of all the components under study.

7.5.2 Building Design

Current design requirements involve the application of loads that are assigned by the building code. The codes often reference ASCE standard 7-88, "Minimum Design Loads for Buildings and Other Structures." As the title suggests, this standard provides guidance for all *structural* loading conditions that apply to buildings, including live, dead, snow, earthquake, wind, roof live, rain, ponding, lateral soil, and fluid loads. The strength degradation due to fire, which could be considered a structural loading condition, is not considered in this design process, and consequently, members are sized to handle only the loads that directly apply to the structure from a building code

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[13] Questions surround the results of this testing. Comparative analysis should be done with discretion.

[14] Ibid.

perspective. If structural elements can safely meet the building code defined loads, they are allowed to be used because they conform to the code requirements.

Other sections of the code apply directly to the fire safety aspects of the building. The major model building codes do not specifically address the structural design and load capacity consequences of fire on a structure or, as a result, firefighter safety issues after a fire has begun within a structure. However, these codes do recognize that standard fire tests that they reference employ representative fire loads for evaluating the comparative performance of structural assemblies.

Fortunately, severe fires seldom occur at the same time as full structural design loading. This will benefit the fire endurance performance of structural members.

7.5.3 Building Codes

No type of construction (e.g., Type I, II, etc.) provides absolute protection for firefighters after building contents have ignited. As noted above, this is especially true from a structural design perspective, since the effects of a fire on structural member performance are not specifically part of the design process. The codes do, however, adjust the size of the building based on occupancy, the potential fire load generated by that occupancy, and the type of construction. Certain features of the building codes aid firefighters in the suppression of building fires and, hence, their safety. These include: fire-rated compartmentation, fire doors, draft- and firestopping, sprinklers, egress, emergency lighting, and other requirements.

In some cases, code requirements are violated during the construction process. This may occur by failing to construct a fire endurance assembly in accordance with the test assembly requirements, or the cutting of holes by HVAC, electrical, or plumbing contractors that violate code provisions or a manufacturer's recommendations. Construction practices that are cause for concern from the point of view of structural and fire performance integrity can be seen on many construction job sites. These situations can contribute significantly to the poor performance of a structural element in an actual fire condition. Improved code enforcement and more widespread code compliance education is needed (e.g., for the builder, electrician, plumber, code official, architect, etc.).

7.5.4 Truss Plate Connections

All steel connections conduct heat into the wood at some point in a fire. Truss plate connections, being steel, do as well. However, they also reflect heat for a period of time during a fire, which protects the wood below those connections. A fire test assembly videotape taken of a small-scale MPC assembly shows the fire performance of a truss plate splice joint under a less-than-design test load following ASTM E119 time/temperature fire conditions.[15] The video distinctly shows the phases a truss plate goes through under fire conditions. Initially, the truss plate reflects radiant fire energy and provides protection to clear wood below the truss plate. This lasts for approximately three minutes (which is not a long period of time), at which point wood below the plate begins to char. The wood ***not protected*** by the plate begins to char approximately one minute into the test, which is two minutes less than the wood that is protected. Once the plate gets hot enough, it conducts heat, and contributes to the charring of wood below the plate and, presumably, around the truss plate teeth. Eventually, charring becomes significant enough that the truss plate loses its holding power and fails. At no time does the plate warp or curl up and fall away from the joint. The time frames found in this testing will not apply to all tests or to actual fires. They only provide a relative comparison of the radiant energy protection that an MPC truss provides. When the char becomes great enough, the load on the truss plate connection causes the wood member to pull away from the truss plate. When this occurs, the plate is often left connected to the other wood member, and does not necessarily fall away from the joint. The existence of this phenomenon is corroborated by tests performed at the Fire Technology Laboratory in Finland, described in **Section 4-3.7** of this report. Here, steel plates are also shown to protect the wood from charring.

There has also been concern over the structural performance of the truss plate, and, consequently, truss performance. Engineering design can make the truss plate connection *structurally* equivalent to many other connection types (e.g., nails, split rings, bolts, etc.).

There has been a very small amount of fire testing performed on the various structural connection types. Engineering design does not take connection fire performance into account. Additional standardized data on the fire performance of connections must be obtained in order to draw any valid conclusions or to be able to perform fire performance-based engineering design with consideration for connections.

7.5.5 Truss Member (Chord or Web) Failure

The concept of one truss member (chord or web) failing, causing an entire truss to fail is *not* an accurate concept for today's lightweight truss construction (e.g., MPC trusses, MPCMW trusses, steel bar joists, etc.). In a theoretical truss, where all joints are a series of frictionless pin-end connected members, this would be the case. In actuality, none of the pre-engineered truss systems are manufactured like this theory suggests. All commonly made trusses today have continuous chords that provide structural continuity and provide a certain amount of additional stiffness; and all connections have a degree of friction and, thus, load carrying capacity. A truss has technically failed when a single member is cut, but cutting a single member by itself will ***not necessarily*** cause catastrophic collapse (see Figure 48 below). In fact, in some cases the truss will still

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[15] Performed by Weyerhaeuser's Fire Technology Laboratory.

carry substantial loads. Total collapse will be dependent on load amount, span, spacing and diaphragm conditions.

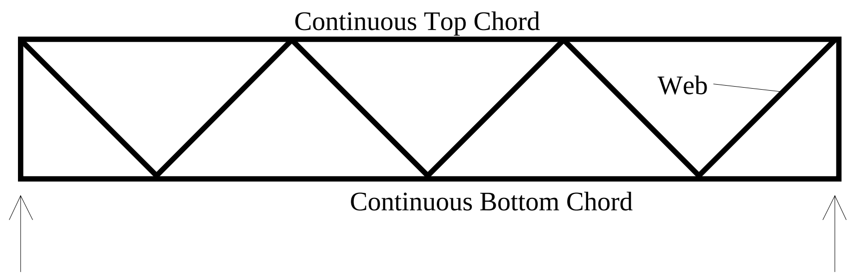

Figure 47. *Standard truss with continuous chords and load-carrying joints similar to MPC trusses or steel bar joists.*

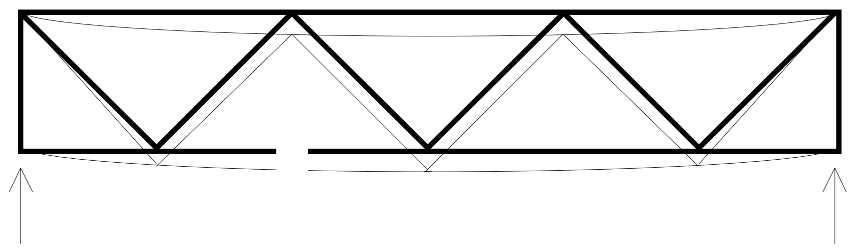

Figure 48. *Standard truss showing deflected shape after the chord is cut. This truss will still carry loads due to the strength capacity of the connections and the continuity of the chords.*

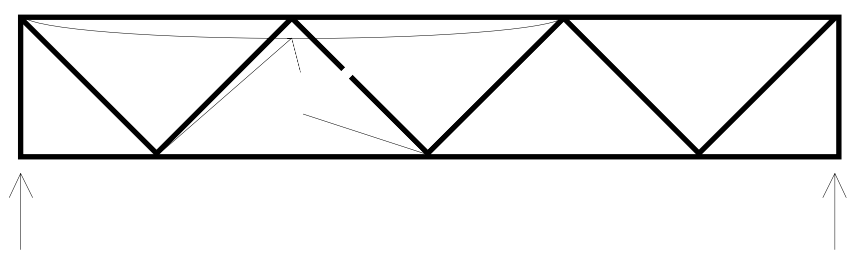

Figure 49. *Standard truss showing deflected shape after the web is cut. This truss will still carry substantial loads.*

For example, if a structural member is spaced two feet on center or less, has a stiff sheathing material (e.g., 5/8-in. plywood) on the top chord, and is braced properly, the system is said to be redundant. When one member fails or is cut through, deformation occurs, the magnitude of which is dependent upon the load applied to the structural system. Under dead load conditions only, cutting through the truss bottom chord would not be noticeable in a redundant system. This is because the structural elements adjacent to the cut member pick up the loads transferred to them through the sheathing material beyond what the cut truss can continue to carry.

The reason that trusses have better load carrying capacity than expected is that most engineered truss types have connections that typically have the capacity to carry loads.

Therefore, by cutting a member in a truss, the load is distributed through the connections to other members that are still sound, which transfers the load to the bearing points. The cut truss will be much more flexible or "spongy" than before it was cut; but the cut will most likely not lead to catastrophic failure.

A forensic videotape clearly shows that cutting the bottom chord of a pitched chord MPC truss does not lead to catastrophic failure, even when the applied load is the weight of two men and the truss spacing is greater than two feet on center (non-redundant conditions).[16]

7.5.6 Girder Versus Redundant Framing Methods

Any structural member that has other structural members framing into it has the potential to cause a large area to collapse under fire conditions. A member to which other members are attached is typically called a girder. If the girder fails in a fire, its failure contributes to the failure of all structural members attached to it. For example, consider a girder truss that is 80 feet long and has eight foot long members framing into it from both sides. Should the girder truss collapse, an area of roof 16 feet wide and 80 feet long (an area of 1280 ft.^{2}) will collapse with it. An example of heavy timber bowstring truss girder framing performance under fire conditions was seen in an automobile dealsership fire in New Jersey[17].

In contrast to this, many lightweight building systems have structural member spacings of two feet on center or less. Using the example above (i.e., 80 ft. span, 2 ft. on center spacing), should one member fail, an area of 320 ft.^{2} could potentially collapse. However, this entire area is less likely to collapse due to the sheathing and lateral bracing that is attached to the members and the load sharing that will take place in the adjacent structural members. This load sharing property is enhanced by a stiff sheathing material (e.g., 5/8-in. plywood) that interacts with the structural elements, creating a diaphragm. In certain cases, a single structural element could completely fail and would continue to be held in place by the structural members adjacent to it, because of the stiffness of sheathing element and continuous lateral bracing.

In some instances, chord and web members have actually failed in existing buildings, and the roof or floor structural system withstood the dead and live loads that were being applied at the time. In a few cases, there have been more than one fractured truss member found within the same truss. Several fractured trusses were also found within the same roof system. This shows that the strength of the roof or floor assembly, including the plywood sheathing and gypsum ceiling, is much greater than the strength of

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[16] Truss load tests at Fish Building Supply Company by Stadelman Engineering, Inc., Menominee Falls, WI, April 20, 27 and 28, 1985 and May 12, 1989.

[17] Klem, Thomas J., **Summary Investigation Report – Five Fire Fighter Fatalities, Hackensack New Jersey,** Fire Investigations Division, National Fire Protection Association, July 1, 1988.

an individual structural element or connection that makes it up. It also reinforces the concept of structural member load sharing or redundancy.

The only situation in which a larger area would fail under redundantly framed conditions would be when several structural elements reach their point of failure at approximately the same time. This phenomenon *doesn't* occur in protected ASTM E119 tests that have been witnessed.[18] Here, one member fails first, which usually begins progressive failure. It may occur in actual fire situations, however, which could be the reason for some of the pancake type failures seen with lightweight building components. Another possible explanation would be the simultaneous failure of the bearing connections, which has to do with the structural integrity and fire performance of the wall system and end connections.

There are no data on the fire performance of girder structural framing contrasted to redundant structural framing. There are also no data on pancake failures of whole floor or roof systems, or its cause. These areas could be developed more thoroughly.

7.5.7 Wooden I-Joist Performance

One of the concerns over I-joist performance is with the use of adhesives in their manufacture. The adhesives used are thermosetting adhesives designed to be very durable when exposed to high moisture conditions. Additional discussion on this is found in **Section 7.5.15**.

There is very little test data available on the fire endurance performance of I-joists. Unsheathed testing is limited to non-standardized and semi-standardized ad hoc tests that can only shed limited light on fire performance characteristics of these products. From this testing, though, it can be surmised that I-joists will perform less well under fire conditions than an equivalent sized solid-sawn member. This is intuitively obvious, given the differences in the cross sections of each. Solid-sawn sections have more "fat" to burn through. To gain fire performance information that will provede the needed information for firefighting tactics, only standardized testing should be performed.

There is one test using 5/8 in. Type C gypsum wallboard directly applied to the bottom flanges of I-joists. This test shows a fire endurance performance of 48 min., which yields an assembly rating of 45 min. This assembly rating is typical for 5/8 in. Type C gypsum wallboard directly applied to engineered components like I-joists or trusses.

7.5.8 Concealed Spaces

The concealed space issue is very important from a firefighting tactics perspective, as it relates to the fire performance of lightweight components. Different components react

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[18] The authors of this report have witnessed greater than 15 ASTM E119 tests, and been involved in writing the reports for more than 30.

differently under identical fire scenarios. This can create serious suppression problems if the building does not have a complete pre-fire plan. Concealed spaces are of particular concern for open web component systems in either a floor or roof. For all truss-type construction (wood trusses, steel bar joists, etc.), a floor system concealed space is a wide open area that fire, heat and smoke can easily move through. When comparing this with solid-sawn (2 x 10) joist construction, wooden I-joists, or steel C-joists that have *no* holes for HVAC or plumbing, and have a direct applied ceiling, a solid web provides better protection against the lateral spread of fire. However, the performance of solid web members is more like open web construction when the web is penetrated for HVAC, electrical, or plumbing. When a dropped ceiling is used, it creates a concealed space for *all* lightweight building components.

In roof construction, all pitched roof systems have an open area through which smoke, heat and fire can move without encumbrance. Flat roof systems, however, are just like floors as described above.

The *major* issue surrounding concealed space fire performance and the spread of fire is the application of code-conforming fire- and draftstopping. If these methods for preventing fire spread are not applied, misapplied, or cut through, the building becomes more vulnerable to structural element and total building collapse caused by fire. Therefore, thorough building inspection is important, that plumbing, electrical, and HVAC holes are sealed and that there is ongoing education on the importance of following fire- and draftstopping requirements and making fire safe penetrations.

Consideration must also be given to the adequacy of the current code requirements for fire- and draftstopping. Changes may have to be made in the codes if they are found to be inadequate. No information is available detailing the adequacy or inadequacy of the code fire- and draftstopping requirements. This needs to be developed.

As noted in **Chapter 3: Fire Loss Statistics**, the majority of residential (49.1%) and apartment (70.9%) fires begin in living areas that are compartmentalized. Only 3.1% and 0.70%, respectively, begin in a structural concealed space, floor, or roof assembly. This reinforces the point that sound compartmentation practices in buildings will contain many fires to a local area, where it can be suppressed most easily. Thoughtless penetration of the compartment or fire- and draftstopping will only aid in earlier fire performance failure of structural building components.

Finally, concealed spaces within roof assemblies (attics) may be used by the building occupant for storage.[19] This results in two problems: the structural element may not be designed for this storage load, which creates a more highly stressed structural member than the design allowed for or is expected; and stored items can also become projectiles once a fire begins, falling through the ceiling to the ground and injuring firefighters

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[19] The automobile dealership fire in Hackensack, New Jersey, is an example of a situation where a concealed space was loaded by storing automobile parts and combustible products in the truss space. See footnotes 28-30 on Page 204 for references.

during fireground operations. The extra loading applied will be a factor in how long the structural member remains in place during the fire. This can contribute to a collapse that is *faster*, or more extensive, than would normally be expected.

7.5.9 Surface Burning Area

There is greater surface burning area in an MPC wood web truss or a wooden I-joist when compared to the surface burning area of a solid sawn joist. For example, the surface area of a 2 x 10 joist is 25 in.^{2} per inch of length. The surface area of a typical 10 in. deep parallel chord truss measured through a diagonal web is 30 in.^{2} per inch of length. A typical 9-1/2 in. I-joist has a surface area of 25.25 in.^{2} per inch of length. Given this, trusses have the greatest amount of surface area to burn, yet this doesn't necessarily predict poorer fire endurance performance.

As noted In **Section 7.5.1**, when discussing mass effects, 2 x 10 joists and 12-inch deep trusses provide roughly the same unsheathed fire endurance performance time, which leads one to conclude that the greater surface area of the truss did not play a major role in relative fire endurance performance. This conclusion may not apply to other lightweight components, and certainly would not apply to heavy timber components. There is no known testing that relates the effect of surface burning to the ultimate fire endurance performance of the various products under study. Data on this relationship will have to be developed to evaluate this in more depth.

7.5.10 Wood Char Rate

The charring rate of wood is a valuable fire protection feature of this engineering material. Under **ASTM E119** fire test exposures, wood ignites in approximately two minutes. Charring then proceeds at a rate of approximately 1/30 in. per minute for the next eight minutes. Thereafter, the char layer has an insulating effect, and the rate decreases to 1/40 in. per minute.[20] With this information, one can often calculate the approximate time that a wood-based system will fail under standard ASTM E119 fire exposures. A graphic example of the effect that charring has on a 2 x 4 member follows in Figure 50.

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[20] Forest Products Laboratory, "Wood Handbook: Wood as an Engineering Material," Agricultural Handbook 72, Washington D.C., U.S. Department of Agriculture, revised 1989, p. 15-3.

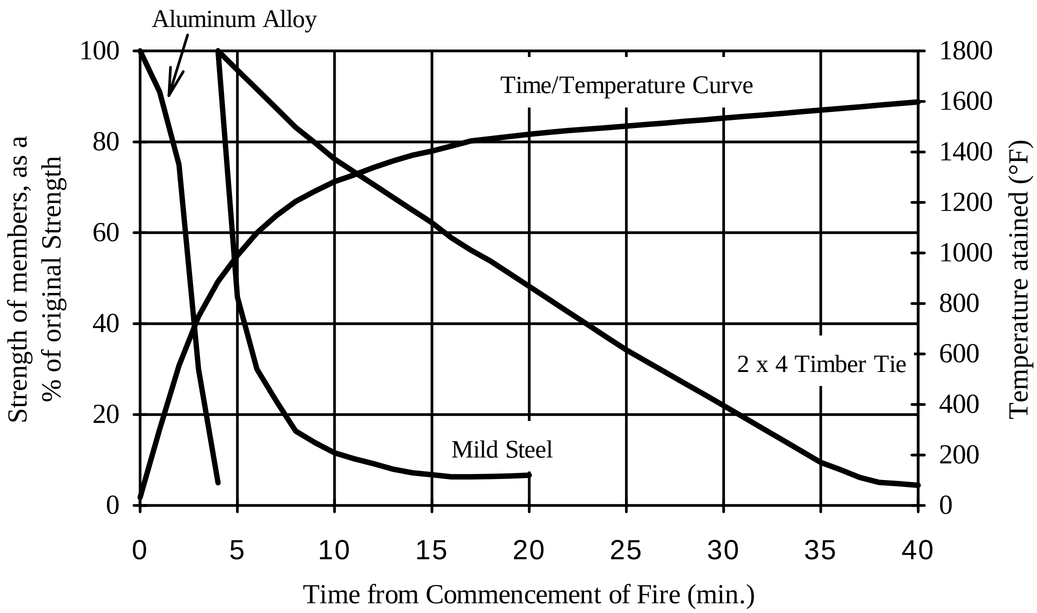

Figure 50. Strength of Steel, Aluminum, and Timber in Relation to the Standard Fire Test. AITC Data[21]

The char rate information shows why heavy timber construction performs so well under fire conditions, and has a separate code classification. It takes a long period of time to burn a 6 in. x 14 in. wood member to a point where it is unable to sustain its load.

7.5.11 Balcony Design

Frank Brannigan brings out a very important point about structural member layout safety. When a structural member is continuous, supporting the living area and the outside common balcony area, any fire that enters the structural compartment will weaken the member supporting the balcony. Since the member is continuous, the concealed space may also allow for spread of the fire into the balcony area if not properly firestopped. This creates a potentially dangerous situation for occupants trying to exit and firefighters trying to enter and exit. This particular construction practice ought to be thoroughly evaluated from a fire performance perspective. Specific installation and fire- and draftstopping recommendations for this application condition should be made.

7.5.12 Truss Collapse

The literature reporting actual fire experience (reviewed in **Chapter 2**) suggests that trusses collapse without warning, and that multiple truss collapses are the rule, not the exception. Reports on actual fires state that there have been cases where truss roofs and floors have collapsed 10 to 15 minutes after the arrival of the fire department. These

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[21] Dock & Harbour Authority, London, England, "What About Fire?", American Institute of Timber Construction, 1972, p. 3.

collapses include both wood- and steel-based lightweight components. In other known cases, trusses and other lightweight components have had endurance times of greater than 30 minutes, or may not collapse at all. The collapse experience that is observed must be viewed in the context of the individual fire scenario. A few questions that need to be asked include:

- How long was the fire burning before the firefighters arrived?
- How heavy was the fire load within the building?
- How heavily was the assembly loaded (dead/live load conditions)?
- Did the roof self-vent?
- Did the structure use redundant member construction or girder construction?
- Were there any warning signals that indicated the potential for a collapse?
- Was there a pre-fire plan in place to determine the type of construction involved?

7.5.13 Collapse Warning Signals

Warning signals prior to collapse is a difficult subject, because they may or may not be present in every fire situation. Recognition of a warning signal is dependent on a firefighter being in the correct place to recognize the signal, and being able to warn others. Some of the warning signals known about for lightweight building construction include:

- A spongy feeling to the floor or roof.
- Floor sag.
- Fire burning through the exterior siding at the floor level, indicating the floor concealed space is on fire. This could also apply to the roof concealed space.

Clearly, this is an area of great concern and has perhaps the greatest life safety implication for the fire service. If there are key warning signals that predict when a collapse will take place, tactics can be developed that will aid in recognition of these warning signals, and allow the fire service to fight a fire with increased safety. The key to this is developing a solid base of information on warning signals that are visible or audible prior to collapse. (there is currently very little information—other than word-of-mouth experience from firefighters—that provides guidance in this area.) This needs to be developed more fully so that firefighter safety on the fireground can be improved, because there may be many collapse warning signals that aren't currently recognized.

7.5.14 Long-Term Truss Performance

A study was performed by the Forest Products Laboratory in Madison, Wisconsin, on the long-term strength performance of a variety of wooden truss types.[22] The conclusion drawn from ten years of long-term loading of these trusses was:

> *There appeared to be no appreciable effect upon strength and stiffness as determined by laboratory evaluation after five and ten years of exposure, with the exception of the nailed plywood gusset truss rafters, which had a 30% reduction in stiffness. All the truss rafters still met acceptable short-term performance criteria.*

From a practical standpoint, there are residential and commercial structures that were built with MPC trusses that are now 30 years old that have had no field performance problems. In fact, there are examples of satisfactory long term field performance for all components reviewed in this study.

7.5.15 Steel Structural Member Performance

> *Steel is non-combustible and does not contribute fuel to the fire, which often leads to unwarranted confidence in its fire-resistance properties. Like all engineering materials, it has structural properties that react adversely to high temperature conditions. Steel loses approximately 35% of its original yield strength and modulus of elasticity at 1000 °F. Steel also has high thermal conductivity, which means it transfers heat away from a localized heat source very quickly. This property, along with its thermal capacity, allows steel to act as a heat sink. When steel can transfer heat to cooler regions, it can take a long time for a member to reach a critical temperature. However, an intense fire that distributes heat evenly along a steel member will reduce this time considerably.*
>
> *Mass and surface area are the most significant factors in determining the fire endurance performance of steel. Heavy, thicker sections have greater resistance to fire than do lighter, thinner ones. Unprotected lightweight sections like those found in bar joists can collapse after five to ten minutes of exposure. Steel's high coefficient of expansion may also cause problems under fire exposure by buckling, twisting, and causing lateral movement in structural elements it is attached to. As an example, a 50-ft. long steel beam heated uniformly to 972° F will increase in length 3.9 in.*[23]

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[22] "Longtime Performance of Trussed Rafters With Different Connection Systems: 10-Year Evaluation," U.S. Department of Agriculture Forest Service, Forest Products Laboratory, Madison, WI, Research Paper FPL 204, Revised 1978.

[23] National Fire Protection Association, "Fire Protection Handbook," Quincy, MA, 1991, pp. 6-62 - 6-66.

Finally, due to steel's high strength, members are often spaced at wide intervals (i.e., six feet or more on center, which typically constitutes a non-redundant condition). This condition requires special consideration during fireground operations so that safe operating conditions are maintained.

7.5.16 Adhesive Fire Performance

The premise that adhesives soften during a fire is erroneous. The adhesives used in engineered wood components (I-joists, LVL, glulam beams, etc.) are typically thermo-setting adhesives that do not soften when subjected to high temperatures. In fact, they get harder. Most often, these adhesives are formulated for durability and resistance to delamination when placed in exterior exposure conditions (i.e., outdoors). These adhesives are typically phenol-formaldehyde or phenol-resorcinol based, and have a char rate that is equal to or better than that of the wood they are bonding.[24] Generally, these adhesives do not ignite at the bond line, but do pyrolyze. Glue laminated beams using these adhesives types are used under heavy timber code classifications, which means they have been proven to have extremely good fire endurance performance behavior.[25]

7.5.17 Fire Testing

There are currently no standardized fire tests that replicate realistic or actual fire scene conditions from a fire service perspective. The test used most frequently to assess the comparative fire endurance performance of building assemblies is ASTM E119. However, this test method is not intended to predict performance times in actual fire situations. The rated time period (e.g., 1-hour) is relative, not absolute. It is not viewed as reliable in predicting realistic fire endurance performance of structural components by the fire safety community. This is clearly an area where agreement on a standardized approach to testing could provide firefighters with additional knowledge. Such information on warning signals and failure modes is needed to develop firefighting tactics in buildings constructed with specific types of lightweight building components. Refining test methodologies could aid in the assessment of the performance of lightweight components in the following areas:

- Warning signals prior to failure.
- Redundant versus non-redundant system fire endurance performance.
- Performance after a ventilation hole is cut into the system.

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[24] Schaffer, E.L. and River, B., conversation on fire performance of adhesives, Forest Products Laboratory, May, 1992, Madison, WI.

[25] "Design of One-Hour Fire Resistive Wood Members (6-inch Nominal or Greater)", Council of American Building Officials Report No NER-250, NFiPA.

- Firefighter safety when operating on assemblies near vent holes or near the fire area.
- Various modes of failure for a structural element, connection, or entire structural system.
- Structural element fire performance strength decay in vented versus unvented structures.
- Modes of failure for concealed space fires.
- The effects of draft- and/or firestopping on assembly fire endurance performance.
- The effects of membrane and fire- and draftstopping penetrations.
- The effects of various fire intensities, replicating as accurately as possible realistic fireground fire conditions and fire growth.

7.5.18 Firefighting Tactics

Firefighting tactics are influenced greatly by the general tenets of firefighting, which are:

- Firefighters are expected to rescue trapped occupants in buildings.
- Firefighters are expected to confine a fire to the area of its origin in most cases.
- Firefighters are expected to extinguish a fire with the least possible damage to the building/contents.
- Interior firefighting is the most effective and efficient method of fire extinguishment.
- Firefighters expect a building (including its components) to perform adequately in order for them to perform their duties. The building must remain intact for a reasonable period of time after firefighter arrival.[26]

Given these tenets, the fire service has developed some initial strategies (found in the literature) to address changes taking place in the design and use of lightweight building components within structural systems. However, knowledge in this area needs to be expanded. Firefighters have recognized that the tactics used to suppress fires may have to change when these components are used. The ability to recognize the structural systems and determine the best tactical approaches is extremely difficult. As a start, this process should include:

- Active pre-fire planning for each building in the jurisdiction, particularly during the initial construction process. Clearly, the more that is known about a building, the easier it will be to fight a fire in it.
- Changes to ventilation procedures.

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[26] Corbett, G.P., "Lightweight Wood Trusses and Fire Notes," March 30, 1992.

- Opening up concealed spaces quickly.
- Being aware of the time factor by always asking, "How long has the fire been burning?" prior to arrival and while on the fireground.
- Being aware of warning signals of impending collapse, and communicating information frequently to fireground command.

> *All assemblies can pose serious dangers to firefighters.* Any *assembly can be fatal if the proper ratios of fire load, time, construction type, penetrations to compartments, and fire- or draftstopping are combined. The amount of time remaining to failure cannot be predicted for* any *assembly type, and should not be attempted on the fireground. Finally, it is a fact that* any *assembly can be dangerous and collapse unpredictably during the early stages of a fire.*[27]

7.5.19 Education and Training

Education and training may be the single most important short- and long-term activity that the fire service can immediately undertake to enhance life-safety on the fire scene. The lightweight component industry must recognize their important role in this educational process. Information about their products and their structural and fire endurance performance must be communicated to the fire service. It follows that the fire community must take facts—those currently available, as well as those which will be ascertained cooperatively—and integrate them into their training programs, pre-fire plans, and tactics.

7.6 Lightweight Component Industry Perspective

The lightweight component construction ndustry is concerned that the negative attitudes that exist toward such construction only serve to creat conflict despite the fact that the components and assemblies conform to the current model code requirements for building construction. When facts regarding engineered products are misunderstood or when the products are blamed for firefighter deaths without thorough analysis, the conflict is increased.

Some deaths and injuries have occurred because fire performance characteristics of different construction systems were not recognized. Because firefighters may have difficulty recognizing that particular structural systems are incorporated into the construction of a burning building, a pre-fire plan would be beneficial for quick assessment.

For example, an automobile dealership fire in New Jersey, where truss failure was implicated in the deaths of five firefighters, did not use lightweight components as part of

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[27] Mittendorf, J. and Brannigan, F., "The Timber Truss: Two Points of View," Fire Engineering, May, 1991.

the contruction. This building was constructed in the late 1940's using bowstring wood trusses, and was renovated extensively in 1973. Reports indicate that the collapse took place between 30 and 45 mintues after the fire alarm office received report of the fire. Changes to the use of the building, inappropriate storage of combustible materials in the truss space, building alterations, mixed use, lack of effective communications on the fireground, and other unforeseen elements at the fire scene all contributed to this loss of life.[28,29,30] To place the responsibility for this incident on the fact that trusses were the major structural element is a gross oversimplification.

Currently, all lightweight building components must comply with applicable building code requirements. In complying, there is an expectation that these products will be allowed to be used in any structure where they meet the intent of the code, where sound engineering principles are utilized, and are in demand by consumers. It is important that products be economical to use in building construction, yet effective where life-safety is an issue. Currently, life-safety issues raised with respect to lightweight building construction appear to be limited to firefighter safety. The safety issues arise due to the general tenets of firefighting given above. This is definitely a concern, but one that is not addressed by the model building codes. There are currently no performance requirements that lightweight components must meet that take into account fireground safety issues.

The manufacturing industry wants to work with the fire service to address product fire performance issues. This should be done in a factual, systematic, and standardized way in order for the lightweight component industry to embrace and promote any measures that may be developed. The best outcome will result when such measures are developed and implemented cooperatively through a consensus process.

[28] Demers, P.R. and David, P., Fire Incident Analysis Five Firefighter Fatalities. Hackensack, New Jersey, July 1, 1988. Prepared for International Association of Firefighters.

[29] Klem, Thomas J., **Summary Investigation Report – Five Fire Fighter Fatalities, Hackensack New Jersey,** Fire Investigations Division, National Fire Protection Association, July 1, 1988

[30] Corbett, Glenn P., "Five Fall in Hackensack," **Fire Engineering,** October 1988

Chapter 8: Conclusions and Recommendations

8.1 Conclusions

The preceding chapters of this report have sought to review the readily available literature found during the literature search, digest the information, concisely report on its content, and then analyze it for accuracy and relevance. As a result of this process, the following conclusions have been drawn:

- Lightweight building components are the structural elements of the future and the trend is for their use to increase. As concern increases over environmental impacts of products and the dwindling natural resources that are available, the use of lightweight building products like those described in this report will only increase, because of their efficient use of valuable natural resources.

- Progress on increasing fire ground safety will be made with continued education and training of the fire safety community. Articles in the firefighting literature should encourage increased learning about building construction. This educational process should incorporate all of the engineering and fire performance facts available. This is an area where lightweight component manufacturers should work with the fire community so that all relevant technology is transferred as factually as the current state of knowledge will allow.

- Engineered products, as the name implies, are highly engineered. The purpose of structural engineering is to provide structural elements that can carry expected loads safely while at the same time, be manufactured economically and use engineering materials efficiently. This is precisely why engineered products (e.g., bar joists, trusses, I-joists, etc.) are so often used as structural elements.

  Associated with this, structural engineering design and code requirements (e.g., ASCE 7-88 Minimum Design Loads for Buildings and Other Structures) do not factor increased loading due to fire degradation of the structural member into design procedures. This means that engineered components are made of lightweight materials that, when combined through engineering analysis, have very high strengths under gravity loads, but not necessarily under attack by fire. This has ramifications on fire endurance performance and, consequently, on the fire service.

- Product mass and surface burning area definitely influence the fire endurance performance of products when one compares a large cross-sectional beam versus a lightweight beam of any material. These effects are dramatically reduced when comparing materials having similar mass and surface area. The key to evaluating the effects of mass and surface area lie in analyzing the components that are effectively resisting fire degradation. For example, in evaluating a 2 x 10 joist and a 2 x 4 truss, the key fire performance resistance dimension is 1.5 in. Test results show similar fire endurance performance of these two products. The critical dimension usually degrades at a similar rate.

- The fire safety community has stated that is has experienced fire scenarios where lightweight building construction structural elements have collapsed more rapidly than would typically be expected. This has led to a major concern over the fire performance of these products.[1] There is a real need to learn as much as possible about the fire performance of the products under study—particularly modes of failure and observable/audible warning signals prior to collapse—so that fireground tactics can be changed and fireground safety enhanced.
- Determining the warning signals of lightweight components prior to collapse is a subject area that needs much more research and development. There may be collapse warning signals available that aren't currently recognized, and there may be situations where no warning signals are present. There is not a large body of information to work with to evaluate this effectively.
- The standardized comparative testing of unsheathed assemblies to date is limited to 2 x 10 joists, MPC trusses, and Steel C-joists as shown in Table 22 in **Chapter 4-1**, page 108. There are no tests available for wooden I-joists, MPCMW trusses, PECSW trusses, and steel bar joists. Standardized comparative tests do exist for protected assemblies for all lightweight components, due to model code requirements. The codes do not require this for the application of unsheathed assemblies.
- Open webbed truss-type components are very useful in construction for running HVAC ductwork, plumbing, and electrical distribution systems through the structural assembly. However, this construction method results in a concealed space when a ceiling is applied. The same is true for I-joists and solid-sawn joists when holes are drilled through these systems, although the spread of fire and gases will not be as rapid. A dropped ceiling creates a concealed space condition for all lightweight building systems.

  The concealed space issue is very important. Different components react differently under identical fire conditions. This can create serious suppression problems. A thorough pre-fire plan can be instrumental in the successful suppression of the fire and aid in fire ground safety.

  Concealed space fire performance will be a concern where structural elements are continuous (and not firestopped) and support the living area as well as an outside balcony area. This creates a dangerous situation by potentially weakening the balcony, which may be the only means of entry and egress for occupants and firefighters. This situation should be thoroughly evaluated, and proper firestopping requirements implemented.

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[1] There is a real need for in-depth documentation of fast collapse fire scenarios so that these situations can be thoroughly evaluated for solutions. The literature does not have much detailed information on this.

The key to preventing spread of fire in a concealed space is to apply code-complying fire- and draftstopping. A major problem, however, is a lack of construction of this fire- and draftstopping, or its penetration by the HVAC, plumbing, or electrical trades. Once penetrated, the fire- and draftstopping ceases to be of value, and will allow fire to spread to other areas of the building unchecked. A major educational effort should be undertaken within code enforcement bodies and construction trades so that the importance of applying code complying fire safety requirements are reinforced and implemented.

- Trusses built today are not built in accordance with frictionless pin-end connection theory, upon which theoretical truss design is based. The chords are often continuous, and the connections at the web member locations often transfer substantial amounts of load. This means that when one member of a truss is cut, whether it be a chord or web, the truss will generally not collapse, even under relatively high load conditions.

  The same concept is true for connections. Should one connection fail, in general, the entire truss will not collapse. Rather, the load is redistributed to other load-carrying elements and connections. Depending on the size of the load, a truss will deflect abnormally, signaling the existence of a structural problem.

  However, it must be remembered that a cut truss has failed in that it will probably not support the full design load. It won't necessarily collapse, however.

- There is a difference in structural failure performance for systems that use framing that is non-redundant as opposed to redundant. The non-redundant systems—often called girder systems—are more complex in their fire response. Should a girder system fail, a large area of roof or floor supported by the girder may fail with it. Girders also carry higher load levels, which means that the structural members that make up the girder are often of larger dimensions. Heavy timber trusses, for example, are typically employed as girders with smaller members framing into them to support a roof or ceiling. (The automobile Dealership in New Jersey, which was constructed with 78-foot-long bow-string, segmental trusses spaced approximately 16 feet on center is an example of girder construction. These trusses were high load carrying, non-redundant structural members. Thus, the failure of a single truss would cause a large section of the roof to collapse. As noted in one report, the collapse "was not solely a function of the fire burning the truss. It was, rather, a result of the combination of fire, heavy structural load (stored auto parts) and, possibly, water that may have collected in the truss loft."[2,3,4])

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[2] Demers, P.R. and David, P., Fire Incident Analysis Five Firefigher Fatalities. Hackensack, New Jersey, July 1, 1988. Prepared for International Association of Firefighers.

[3] Klem, Thomas J., Summary **Investigation Report – Five Fire Fighter Fatalities, Hackensack New Jersey,** Fire Investigations Division, National Fire Protection Association, July 1, 1988.

[4] Corbett, Glenn P., “Five Fall in Hackensace,” **Fire Engineering,** October 1988

Structurally redundant systems typically have elements spaced 2 feet on center or less, and carry much lighter loads. A system is redundant if adjacent structural members can be expected to share load. For instance, if one member fails, the two adjacent members will pick up the load originally resting on the failed member. Much of the load sharing capability comes from the sheathing diaphragm; therefore, even greater on-center spacings may share load and be considered redundant, depending on the strength of this sheathing material and the resulting diaphragm. As spacings become greater than 2 feet on center, the systems will typically have increasingly less structural member redundancy, and begin to fall into a girder classification.

Understanding the difference between these two framing methods and, if possible, recognizing the construction type in the building will benefit fireground command decision-making capabilities.

- Truss plate connectors do reflect radiant fire energy during the initial phases of a fire and then progress into a conduction phase that results in charring below the plate and the eventual degradation of the strength of the joint. The fire will not cause the plate to pull or curl away from the joint, but the load on the wood members will.

  Where these connections have been fire tested; fire endurance performance is less than 10 minutes, based on the small amount of test data available. Additional standardized data on the fire performance of connections must be obtained in order to draw any valid conclusions.

- The charring rate of wood can be beneficial to the fire endurance performance of all wood-based systems. The char layer acts as an insulator of uncharred wood below the surface, allowing the wood to continue to carry the applied loading until the char layer becomes too deep, and the applied stress causes member failure. This process is more significant in large cross-sectional members than in small cross-sectional members. The concepts apply to all wood members, however. It is because of the wood charring process that there is a section in most building codes on the use of heavy timber.

- There is very little data available to make an accurate assessment of the fire performance of wooden I-joists. The reduced cross-section of the I-joist causes concern within the fire service because the fire performance will not be the same as a traditional joist (intuitively, it will be less). The exact differences in fire performance are not known. However, I-joists have been tested using gypsum wallboard protection. They have been found to perform similarly to other engineered products (e.g., trusses) when gypsum protection is directly applied.

- The adhesives used in engineered components are typically thermosetting adhesives which do not soften or lose their bonding capabilities during a fire. In fact, under heat, the bond becomes stronger. Under fire conditions, the adhesives will char in a manner similar to that of solid wood.

- Each type of construction in the building code allows the use of unprotected non-fire resistance rated structural systems. The codes do recognize the increased possibility of fire damage in unprotected buildings, and restrict the allowable areas and heights for this construction. As protection is installed, allowable areas and heights increase.

- Building code provisions are developed so occupants can evacuate safely, and so that the fire service has adequate access (clear path for trucks, etc.) to the building to suppress a fire. The focus of building codes is not to protect thos who enter a bulding once a fire has gotten out of control in a building.
- All changes that are made in building code requirements (both model and local) should be based on technically valid substantiating evidence. Without this, costly, ineffective and technically unsound public policy decisions may be made.
- Lightweight engineered trusses and other composite engineered products have a solid history of structural performance in field applications in the construction industry. In all likelihood, there are several billion steel and wood trusses still supporting their applied loads in construction completed over the last 30 to 40 years.
- The general tenets of fighting fires places firefighters at risk and influence the strategy tactics used on the fireground. The fire service has proposed some changes in procedures that could help prevent disasters at the fire scene. These include:
  - Pre-planning all structures.
  - Venting the roof using only proper safety precautions.
  - Opening concealed spaces quickly to determine current fire location.
  - Being aware of the time factor by always asking, "How long has the fire been burning?" prior to arrival and while on the fireground.
  - Communicating all abnormalities to fireground command.
  - Watching for indications of structural deterioration.
  - Broadly disseminating new tactical safety concepts learned from each fire.

  This is only the beginning, however. With help from the industry, new tactical procedures must be developed continuously, taking into account new construction methods, to increase fireground safety.
- Education and training may be the single-most important collective activity the fire safety community and lightweight building products industry can jointly immediately undertake to enhance life safety on the fire scene.
- Statistics reviewed for this report suggest that the firefighter life safety efforts ought to be on protected lightweight building construction elements. This is due to the fact that the majority of fires begin in compartmentalized spaces. There is some experiential data, however, that suggest that there may be a life safety issue with unsheathed (i.e., unprotected) lightweight building assemblies and when these assemblies support sprinkler systems. There is not a good statistical base of information to corroborate the experiential data, however.

Since most fires begin in compartmentalized living spaces, the addition of smoke detectors and the use of sprinklers will save civilian lives and go a long way toward protecting firefighters on the fireground. Firefighter safety will be enhanced because civilians will be out of the building and/or the fire will be contained and possibly extinguished.

In most sprinkler activations, one head usually controls the fire. Generally, this activation occurs in a room that is compartmentalized (e.g., a protected assembly). Given this, there is no question that the use of functioning and well-maintained sprinklers will reduce life and property loss. There is also the strong possibility that sprinklers will reduce fireground fatalities, as they contain and then extinguish fires prior to the arrival of the fire trucks. The fire safety community is concerned over sprinkler applications where certain building areas are exempt from being sprinklered. Should a fire start in one of these areas, it is difficult to predict what will happen.

A risk assessment should be performed to fully address the risks associated with fatalities directly related to structural member collapse. One firefighter death directly attributable to engineered components is one too many. The focus should be to reduce this risk to as small as is justifiably possible given the relative risks.

- Finally, when available test information on components and sprinklers is reviewed, it reveals a lack of product performance test standards and acceptance criteria for the components under study, under the following conditions:
  - **Fire performance of unsheathed component assemblies:** There is currently no standardized test procedure to evaluate the fire performance of unsheathed components that is acceptable to both the fire service and the building component industry. The fire safety community desires a test procedure that replicates "realistic" fire conditions. Currently, the most widely accepted test procedure is ASTM E119, which uses a standard time/temperature relationship to allow comparison of performance. A consensus standardized test method that incorporated the fire safety community’s need for information would be very beneficial.

    Beyond this is the issue of there being no fire endurance performance requirements for the use of unsheathed fire endurance assemblies mandated by the model code groups. Because of this, no testing being conducted on these assemblies for use in code complying construction. Hence, no large body of test data available on unsheathed fire endurance performance.

    A consensus standard test method that focuses on failure modes and audible/visual warning signals prior to collapse would be useful to improve fireground tactics. Associated with this will be the need for criteria that states what is acceptable product performance. Once this consensus is developed, a body of test data would be available to guide fireground tactical response.
  - **Fire endurance performance of components when a fire originates in a concealed space:** Similar to unsheathed assemblies, there do not appear to be standardized test procedures or performance criteria that can be used to

evaluate the performance of a component when fire begins in a concealed space.[5] There are also no model code requirements that establish acceptable performance. A consensus standard focusing on fire performance in concealed spaces would be useful. Associated with this will be the need for criteria that state what is acceptable performance, so that the tactical response in the majority of cases will be effective.

– **Fire endurance performance of lightweight components that support sprinkler systems:** As with the preceding two points, there are no standardized test procedures or performance criteria that can be used to evaluate the fire endurance performance of structural members supporting sprinklers when a fire begins below a structural member and its attached sprinklers. There are also no acceptance criteria that can be used to determine satisfactory fire endurance performance of lightweight components as they support sprinklers.

  There are, however, standards that have been developed specifically to assess sprinkler head performance. These standards include UL 199, "Automatic Sprinklers for the Fire Protection Service;" UL 1626, "Quick Response Sprinklers for the Fire Protection Service;" and UL 1767, "Early Suppression, Fast Response Sprinklers."

  The application of lightweight components in buildings using sprinkler systems is fully developed by NFPA 13, 13R or 13D. The connection of sprinkler systems to components can easily be performed using traditional engineering calculations or through tests that have been run on connecting systems.

  The express concern, however, is whether the structural elements, in certain applications, will maintain their load carrying capability long enough for sprinklers to contain or extinguish a fire that begins under a sprinkler system or within a concealed space. This is where a consensus test standard and criteria may be needed.

In contrast to the above three points where standardized test procedures and acceptance criteria are not available, is the protected fire endurance compartment. Here, there is a commonly accepted test procedure (ASTM E119) and code-mandated performance criteria for protected assemblies and penetrations of those protected assemblies. Because of this, the lightweight building component industry has developed various assemblies that meet these code requirements.

It appears that agreement on consensus-based standardized testing procedures and performance criteria for the areas defined above is a primary need for both the lightweight building components industry and the fire service, in order to resolve issues

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[5] A test method to measure the performance of an assembly within a concealed space above the ceiling exists in Germany.

surrounding the fire endurance performance of these products. Once standards and acceptance criteria are defined, there will be no question on how the testing should be performed, and what acceptable performance will be. If these standards and performance criteria are established through a partnership between manufacturers and the fire service, all concerns regarding product performance can be addressed. Industry will know the fire performance expectations of its products, and the fire service can formulate the appropriate suppression tactics and fireground strategies based upon this known performance.

8.2 Recommendations

From these concluding comments, the following activities should be undertaken to continue the process of resolving the issues surrounding the fire performance of lightweight building construction:

- Representatives from the fire service, lightweight building component industry, model codes, and other groups should form a committee. This committee can develop initial test protocols and performance criteria that can be used to evaluate unsheathed, concealed space, and sprinkler performance of the lightweight building components described herein. This would entail collecting all testing protocols that may be used or have useful sections, and integrating them into a draft test protocol. Performance criteria could be developed in the same manner. Testing could then be undertaken following these prepared guidelines. The goal would be to begin to fill in the knowledge gaps that have been identified in this literature review and technical analysis with particular emphasis on firefighting safety and the various tactical responses that could be undertaken.
- As noted in the conclusions above, the need for education and training is significant. There is an immediate need for development of several technology transfer activities that would provide factual information surrounding the performance of lightweight engineered building components. The content could include:
  - Engineering principles that apply to these building components.
  - Explanation of the fire performance of building components that are used in construction systems.
  - Explanation of fire endurance testing procedures.
  - Explanation of the use of mathematical fire endurance models as they are developed for construction components.
  - The importance of code-conforming construction, and how violations of fire- and draftstopping influence fire performance of building components.
  - Strategy and tactics that are developed for fighting fires in buildings that employ lightweight building components. This includes developments based on current knowledge, and would include knowledge gained through testing and experience.

– Developing the database technology that would support pre-fire planning. This could then be expanded to gather detailed information on the fire performance of lightweight components in buildings that use them.

With a broad base of fire service and industry support working cooperatively in educational and standards activities as described above, credibility of the work product within the fire service and lightweight engineered product industry would be immediate. This would provide the greatest possible positive impact on knowledge about product fire endurance performance, and hence, general life safety, as well as improve safety for the firefighter on the fireground.

Appendix A: Glossary of Terms

Preface

Pertinent definitions have been taken from the ASTM Standard E176-91d, "Standard Terminology Relating To Fire Standards."

ASTM Standard Definitions

Combustible, adj **–** capable of undergoing combustion.

Discussion: The term combustible is often delimited to specific fire-exposure conditions. For example, building materials are considered combustible if they are capable of undergoing combustion in air at pressures and temperatures that might occur during a fire in a building. Similarly, some materials that are not combustible under such conditions may be combustible when exposed to higher temperatures and pressures, or to an oxygen-enriched environment. Materials that are combustible in bulk form may be combustible in finely divided form.

Fire endurance, n **–** a measure of the elapsed time during which a material or assemblage continues to exhibit fire resistance.

Discussion: As applied to elements of buildings, et shall be measured by the methods and to the criteria defined in Test Methods E119, E152, E163, or E814.

Fire hazard, n **–** the potential for harm associated with fire.

Discussion: A fire may pose one or more types of hazard to people, animals, or property. These hazards are associated with the environment and with a number of fire-test-response characteristics of materials, products, or assemblies including but not limited to ease of ignition, flame spread, rate of heat release, smoke generation and obscuration, toxicity of combustion products, and ease of extinguishment. (1989)

Fireproof, adj **–** an inappropriate and misleading term. Do not use.

E176 Non-Mandatory Commentary: This term was originally used to describe buildings having all non-combustible structural elements and some degree of fire resistance. However the term has been misunderstood to mean an absolute or unconditional property, and therefore the use of the term, fireproof, is inappropriate and misleading. (1985)

Fire resistance, n – the property of a material or assemblage to withstand fire or give protection from it.

Discussion: As applied to elements of buildings, it is characterized by the ability to confine a fire or to continue to perform a given structural function, or both.

Fire resistive, adj – having fire resistance.

Fire-retardant barrier, n – a layer of material which when secured to a combustible material or otherwise interposed between the material and a potential fire source, delays ignition and combustion of the material, when the barrier is exposed to fire.

Fire-retardant coating, n – a fluid-applied surface covering on a combustible material which delays ignition and combustion of the material when the coating is exposed to fire.

Fire risk, n – the probability that a fire will occur and the potential for harm to life and damage to property resulting from its occurrence.

E176 Non-Mandatory Commentary: Fire risk is a quantitative description of the potential for injury or loss. The risk of loss of property will depend upon the probability of an ignition occurring, the fire-test-response and fire performance characteristics of the materials, product, and assemblies in a given situation, and the existence of fire containment or extinguishing systems. Where the risk is that of injury or death, consideration must also be given to the probability of human exposure and the physiological and psychological responses of persons to the fire. Risk is a scalar quantity that may have any one of a range of values, and does not describe the acceptability of that value to an individual or society. Two persons, when presented with the same risk situation, might reach different conclusions relative to their willingness to accept that risk.

Fire test exposure severity, n – a measure of the degree of fire exposure, specifically in connection with test methods E119, E152, and E163, the ration of the area under the curve of the average furnace temperature to the area under the standard time/temperature curve, each from the start of the test to the end or time of failure, and above the base temperature 68˚ F (20˚ C).

Fire-test-response characteristics, n – a response characteristic of a material, product, or assembly, to a prescribed source of heat or flame, under controlled fire conditions; such response characteristics may include, but are not limited to, ease of ignition, flame spread, heat release, mass loss, smoke generation, fire endurance, and toxic potency of smoke.

Discussion: A fire-test-response characteristic can be influenced by variables of exposure, such as ignition source intensity, ventilation, geometry of item or enclosure, humidity, or oxygen concentration. It is not an intrinsic property, such as specific heat, thermal conductivity, or heat of combustion, where the value is independent of test variables.

A fire-test-response characteristic may be described in one of several terms. Smoke generation, for example, may be described as smoke opacity, change of opacity with time, or smoke weight. No quantitative correlation need exist between values of a fire-test-response characteristic for different materials, products, or assemblies, as measured by different methods, or tested under different sets of conditions for a given method.

Flame resistance, n – the ability to withstand flame impingement or give protection from it.

Flame-retardant coating, n – a fluid-applied surface covering on a combustible material which delays ignition and reduces flame spread when the covering is exposed to flame impingement.

Flame-retardant treatment, n – the use of a flame-retardant chemical or a flame-retardant coating.

Flame spread index, n – a number or classification indicating a comparative measure derived from observations made during the progress of the boundary of a zone of flame under defined test conditions.

Ignition, n – the initiation of combustion.

Discussion: The combustion may be evidenced by glow, flame, detonation, or explosion. The combustion may be sustained or transient.

Ignition temperature, n – the lowest temperature at which sustained combustion of a material can be initiated under specified conditions.

Discussion: While the phenomenon of combustion may be transient or sustained, in fire testing practice the ignition temperature is considered reached when combustion continues after the pilot source is removed.

Mass burning rate, n – mass loss per unit time by materials burning under specified conditions.

Noncombustible, n – not combustible.

Pyrolysis, n – process of simultaneous phase and chemical species change caused by heat (compared to **smoldering**).

Smoldering, n – Combustion of a solid without flame, often evidenced by visible smoke.

Discussion – Smoldering can be initiated by small sources of ignition, especially in dusts or fibrous or porous materials, and may persist for an extended period of time after which a flame may be produced.

Standard time/temperature curve, n – in fire testing, a graphical representation derived from prescribed time/temperature relationships, and used to control furnace temperature with progressing time.

Discussion: One example is found in Test Method E119.

Superimposed load, n – force applied to a specimen or structure other than that associated with its own mass.

Technical Advisory Committee (TAC) Definitions

I-Joist Definitions

Wooden I-joist, n – a structural member manufactured using sawn or structural composite lumber flanges and structural panel webs, bonded together with exterior exposure adhesives, forming an "I" cross-sectional shape. These member are primarily used as joists in floor and roof construction.

I-joist flange, n – this material consists of solid sawn lumber or laminated veneer lumber. The lumber is machined in a variety of sizes and then routered for web attachment. The sizes include 1-1/2" by 1-3/4" by 2-1/4", 1-1/2" by 3-1/2" and other sizes per proprietary design. (See figure below.)

I-joist web, n – this material is placed between the flanges of an I-joist. The width of this material varies with the depth of the truss. The web material also comes in a variety of thicknesses which include; 3/8", 7/16", 1/2", etc. The web material may be oriented strandboard, hardboard, plywood, or other proprietary composite material. (See figure below.)

Oriented strandboard, n – A type of particle panel product composed of strand-type flakes which are purposefully aligned in directions which make a panel stronger, stiffer, and with improved dimensional properties in the alignment directions than a panel with random flake orientation.

Plywood, n – A glued wood panel made up of relatively thin layers of veneer with the grain of adjacent layers at right angles, or of veneer in combination with a core of lumber or of reconstituted wood.

Truss Definitions

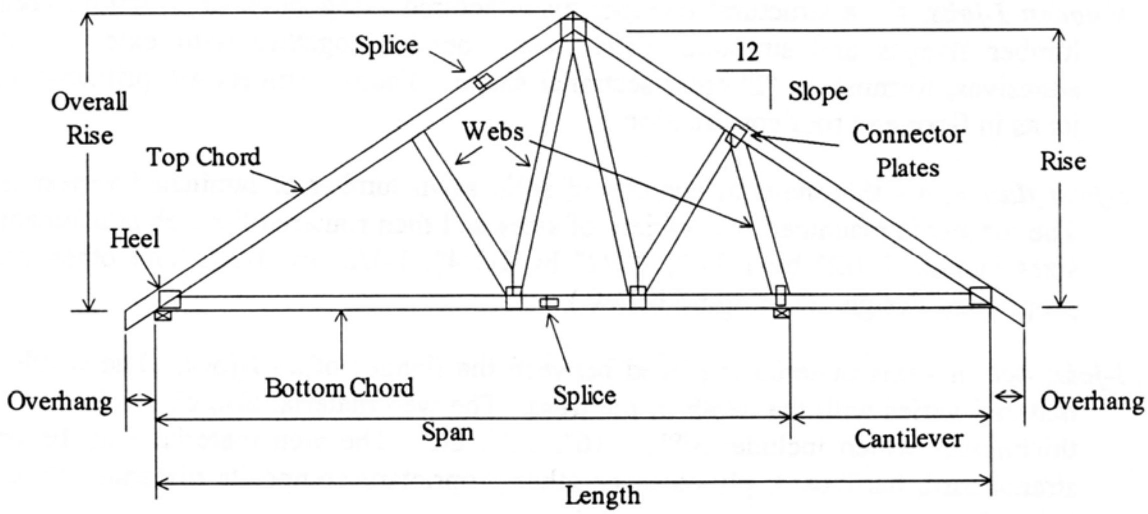
Metal connector plate, n – a connector made from a specific gauge and specific strength steel sheet that is punched with a specific tooth pattern. Each tooth pattern represents a proprietary product. These stamped metal connectors come in a variety of sizes and are pressed into two or more wood members to form a joint which resists axial, moment and eccentrically applied forces.

Metal plate connected (MPC) wood truss, n – a series of dimension lumber members typically assembled to form a series of planar triangles. The chord members are connected to each other and to web members through the use of metal plate connectors.

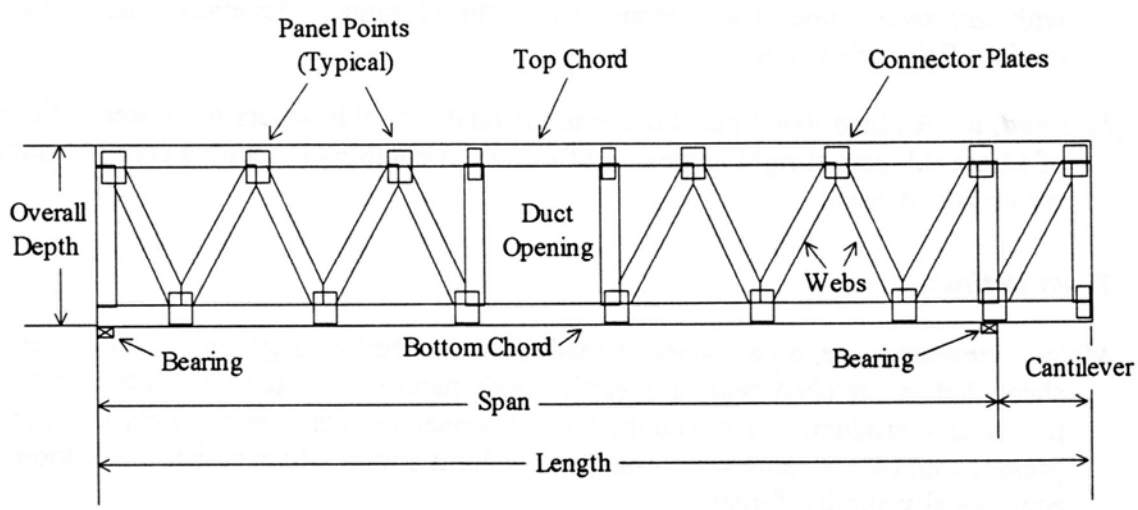
Bottom chord, n – a dimension lumber member that forms the bottom perimeter of the truss. (See figure below.)

Web members, n – dimension lumber members that form the interior members of the truss. (See figure below.)

Pitched chord truss, n – the top chord singly or the top and bottom chords together slope to provide a surface that is at some angel to the horizontal plane. (See figure below.)



Parallel chord truss, n – the chord members of this truss are at a constant distance from each other throughout the length of the truss. (See figure below.)



Pin-end connected metal web (PECMW) truss, n – this truss has steel pins running through wood chords that connect steel webs between the top and bottom chord.

Metal plate connected metal web (MPCMW) truss, n – this truss has wood chords connected with metal webs that have punched metal connectors on the top and bottom of the web.

Metal plate connected metal web (MPCMW) truss, n – this truss has wood chords connected with metal webs that have punched metal connectors on the top and bottom of the web.

Steel bar joist truss, n – this truss is made up of a series of steel top and bottom flanges and steel web members welded together to form a truss.

Steel joist, n – this is a joist of specified gauge and strength that is typically formed into a C-shape and used as a joist or rafter element in floor and roof systems.

Other Definitions

Composite Wood Joist, n – a wooden joist that is made up of composite materials such as waferboard or oriented strandboard for the web material and parallel strand lumber or laminated veneer lumber for chords. This element has dimensions like solid-sawn joists, and is used in a manner similar to solid sawn joists for joists and rafters.

Connection Systems, n – these are the locations within a structure that join one structural element to another. This can include nails, bolts, steel side plates, light gauge metal hangers, bearing clips, etc.

Dimension lumber joists, n – lumber manufactured from the natural wood fiber in trees, cut and dried to nominal dimensions such as 2 x 6, 2 x 8, 2 x 10, 2 x 12, etc., which are used in floor and ceiling systems.

Dimension lumber rafter, n – lumber manufactured from natural wood fiber cut from trees and dried to nominal dimensions such as 2 x 6, 2 x 8, 2 x 10, 2 x 12, etc., which are used in roof systems.

Glue laminated beams, n – a structural element made up of laminations consisting of dimension lumber and/or LVL and/or PSL. The individual laminations are adhesive bonded under heat and pressure and ordered so that a specific composite strength results.

Laminated veneer lumber (LVL), n – a composite of wood veneer sheet elements with wood fibers primarily oriented along the length of the member. Veneer thickness shall not exceed 0.25 inches (6.4 mm) (Per 11th draft of Structural Composite Lumber Standard.)

Light-frame construction, n – any method of construction utilizing dimension lumber joists, MPC trusses, MPCMW trusses, PECMW trusses, steel bar joist trusses, wooden I-joists, or composite wood joists as floor or roof system structural elements.

Parallel strand lumber (PSL), n – a composite of wood strand elements with wood fibers primarily oriented along the length of the member, bonded together under heat and pressure with exterior durable adhesives. The least dimension at the strands shall not exceed 0.25 inches (6.4 mm) and the average length shall be a minimum of 300 times the least dimension.

Structural Composite Lumber, n – Structural composite lumber is either laminated veneer lumber (LVL) or parallel strand lumber (PSL). These materials are intended for structural use and they shall be bonded with an exterior adhesive, qualified in accordance with ASTM D2559 and, in Canada, conforming to the appropriate section of CSA standards for wood adhesives.

Appendix B: Biographies of Fire Service Personnel

REFERENCES IN THIS APPENDIX ARE LISTED IN ALPHABETICAL ORDER BY LAST NAME.

Francis L. Brannigan

Francis L. Brannigan has had a lifetime of varied professional fire protection experience. During World War II he directed a naval fire fighting school, commanded a seagoing fire fighting unit, and served as a district chief in the unique Army-Navy-Pancanal fire protection organization. He remained with the Navy after the war to help develop a competent fire service for the Naval Shore Establishment.

He served for many years as the Public Safety Liaison Officer of the federal Atomic Energy Commission. He developed the Chain Reaction Training Program for fire officers in the correct handling of radiation accidents, and the Fire Loss Management program for the protection of life and property from fire.

At Montgomery College, Rockville, MD, he developed a model Fire Science Program, assembling an outstanding adjunct staff, each a nationally recognized expert in his field. He is currently a member of the adjunct staff of the National Fire Academy, Emmitsburg, MD and the Fire and Rescue Institute, University of Maryland, College Park, MD. In association with his wife, Maurine, he has assembled extensive collections of slides on all aspects of building construction and fire loss management. Jointly they conduct seminars across the country.

He was honored by the Society of Fire Protection Engineers with full membership, despite the fact that his degree was not taken in engineering. He served for many years on National Fire Protection Association Technical committees. He received the Fire Angel Award from the Cleveland firefighters and the Training Officers Conference Award.

The Chesapeake Chapter of the International Society of Fire Service Instructors founded the Francis L. Brannigan Instructor of the Year Award in his honor.

Excerpted from ***Building Construction for the Fire Service,*** *2nd Edition, Francis Brannigan, 1982.*

Allen B. Clark, Jr.

Allen B. Clark, Jr., began his fire service career in Virginia by service on U.S. Forest Service pickup crews. He joined the Bell Township department in 1975 and, after intensive training, became assistant chief and training office in 1979 and chief in 1980.

Referenced from "The Bare Facts on Hidden Dangers, " ***Fire Command Magazine****, July 1984.*

Glenn P. Corbett

Glenn P. Corbett is the administrator of engineering services for the San Antonio [Texas] Fire Department. He has a bachelor's degree in fire science from John Jay College of Criminal Justice in New York City and is working on a graduate degree in fire protection engineering at Worchester Polytechnic Institute in Worchester, Massachusetts. His fire service background includes seven years as a volunteer firefighter in northern New Jersey.

Referenced from "Lightweight Wood Truss Floor Construction: A Fire Lesson, " ***Fire Engineering Magazine,*** *July 1988.*

Bruce E. Cutter

Bruce E. Cutter is a Captain with the Boone County Fire Protection District, Missouri. He is also an Associate Professor of Forestry at the University of Missouri at Columbia where he teaches courses in wood technology, wood utilization and wood engineering.

Excerpted from "Working Together, " ***WoodWords****, May, 1990.*

Vincent Dunn

Vincent Dunn, who has been with the City of New York Fire Department for 31 years, is deputy chief in command of midtown Manhattan, one of the most densely populated areas in the U.S. He holds a master's degree in fire administration and teaches at the National Fire Academy and Manhattan College. Chief Dunn is the author of the text and video seriers **Collapse of Burning Buildings**, published by Fire Engineering Books.

Excerpted from "Firefighter Death and Injury: 50 Causes and Contributing Factors," ***Fire Engineering Magazine,*** *August, 1990.*

John W. Mittendorf

John W. Mittendorf is a battalion chief and 27-year veteran of the Los Angeles City Fire Department. He has an associate's degree in fire science. Chief Mittendorf is the author of the books **Ventilation Methods and Techniques** and **Facing the Promotional Interview,** published by Fire Technology Services.

Excerpted from "The Timber Truss: Two Points of View, " ***Fire Engineering Magazine****, May 1991.*

William Peterson

William Peterson is currently fire chief from the City of Plano, Texas, and is former fire marshal for the City of Bolingbrook, Illinois.

J. Gordon Routley

J. Gordon Routley, a registered professional engineer, is former Assistant to the Fire Chief of Phoenix, Arizona; Fire Chief for the Shreveport, Louisiana Fire Department; and Chair of the Heath and Safety Committee of the International Association of Fire Chiefs. He is also Secretary of NFiPA's Technical Committee on Fire Service Occupational Safety. Among his duties with the Phoenix Fire Department is the on-scene evaluation of the structural integrity of burning buildings. He is currently a consultant for the fire service and Chairperson of the NFPRF Technical Advisory Committee for the National Engineered Lightweight Construction Research Project.

*Excerpted in part from Fire **Journal Magazine**, January/February 1989, p. 83.*

Appendix C: Comparative Risk Statistics

To put the risk of firefighter fatality due to lightweight components into comparative perspective, Table 43 delineates fatalities per year for firefighters, agricultural workers, construction workers, mining workers, and police officers. This will provide a basis upon which to ascertain the level of risk firefighters face in their workplace.

Year	Agricultural[a]	Construction[a]	Mining[a]	Police[b]	Firefighters[c]
1980	2000	2500	500	165	137
1981	1900	2200	600	157	135
1982	1800	2100	600	164	123
1983	1800	2000	500	152	112
1984	1600	2200	600	147	119
1985	1600	2200	500	148	126
1986	1700	2100	400	133	118
1987	1500	2200	200	158	130
1988	1300	2100	300	155	135
1989	1300	2100	300	146	116

a Source: Accident Facts, National Safety Council, 1981-1990 eds.
b Source: Statistical Abstract of the United States, 1991 ed.
c Source: NFPA Journal, August 1991.

Table 43. Fatalities in Selected Fields, 1980-1989

In order to get a better idea of how fatalities compare between these occupations, Table 44 contains the same data normalized to show fatalities per thousand people in each occupation.

Year	Agricultural[a]	Construction[a]	Mining[a]	Police[bc]	Firefighters[ad]
1980	61	45	50	0.38	0.12*
1981	54	40	55	0.35	0.12*
1982	52	40	55	0.37	0.11*
1983	54	37	50	0.34	0.10
1984	46	39	60	0.31	0.11
1985	49	37	50	0.31	0.12
1986	52	33	50	0.28	0.11
1987	49	35	38	0.33	0.12
1988	48	34	25	0.32	0.13
1989	40	32	43	0.29	0.11

a Source of fatality statistics: Accident Facts, National Safety Council, 1981-1990 eds.
b Source of fatality statistics: Statistical Abstract of the United States, 1991 ed.
c Source of total number of police officers: U.S. Department of Justice, Federal Bureau of Investigation, "Crime in the United States," 1980-!989 eds.
d Source of total number of firefighters: Michael J. Karter, Jr., "U.S. Fire department Profile Through 1990," Fire Analysis and Research Division, NFiPA, November 1991.
* Total Number of firefighters was not available for these years. Data were extrapolated.

Table 44. Fatalities in Selected Occupations per 1000 People in Each Occupation in the U.S., 1980-1989

Appendix D: Obtaining the NFPRF Bibliography

The bibliography of articles discussed in Chapter 2 contains approximately 2,000 entries, and can be obtained from the NFPRF by writing to:

NFPRF
1 Batterymarch Park
Quincy, MA 02269-9101

or by calling:

607/770-3000,

and asking for the Research Foundation.

The bibliography comes in two formats: printed or on floppy disk.

THE NATIONAL FIRE PROTECTION RESEARCH FOUNDATION is an Independent public nonprofit Foundation established by the NFPA to Provide practical, usable data on fire risk and state-of-the-art firesafety methods. Since 1982, the Research Foundation has served standards writers, firesafety professionals, corporate and public agency top management. The Foundation brings together research centers of excellence and capital from various directions to focus objective research initiatives on the most crucial fire problems of the day. The Foundation pursues its mission through research in two program areas: Fire Risk Assessment, and New Technologies and Strategies.

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