Structural Engineer's Guide to Fire Protection

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The Strength Behind the Beauty

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Chapter One - Introduction

From the time that Neolithic man first tamed the power of fire to cook his meats and heat his hut, man has lived with the risk of being consumed by its power. Throughout recorded history there are accounts of entire cities being destroyed by fire – ancient Rome, London, Chicago, San Francisco and Dresden. These cities were all rebuilt with more fire resistant construction, but still the threat of fire remained.

Figure 1-1 Chicago 1871 - *National Archives, Washington, D.C.*

Today we have comprehensive Building Codes that regulate the way we design and construct buildings in the interest of guarding public safety and property. The primary focus of Building Codes has always been fire safety with issues such as structural adequacy being a secondary consideration. These codes have served us well. A century has passed since San Francisco was devastated by fire following the 1906 earthquake and no U.S. cities have burned to the ground in that period.

There have been a few significant high-rise building fires in recent decades, the 1988 Interstate Bank fire in Los Angeles, CA and the 1991 Meridian Plaza fire in Philadelphia, PA. In both cases the structures remained standing after the fire was extinguished and the majority of the occupants were able to escape safely. While the structure of the Interstate Bank building sustained minor structural damage and was repaired, the Meridian Plaza structure was extensively damaged and was eventually demolished. In both cases the Building Code objectives of protecting the safety of building occupants had been achieved. These events served to reinforce a sense of complacency in the Architectural and Engineering communities that we were doing all of the right things.

On September 11, 2001, our world changed. The terrorist attack on the World Trade Center towers was a tragedy that few will forget. There was a loss of innocence on that day. One of the victims was our confidence in the way we protect building occupants and structures from uncontrolled fire in tall buildings.

Fire Protection Systems

Modern buildings employ multiple fire protection systems. Active systems such as sprinkler systems are intended to control a developing fire. Detection and alarm systems are designed to provide early warning to building occupants and firefighters of a fire. Manual systems, including standpipes, hose cabinets and fire extinguishers assist firefighters in fighting fires. Egress systems allow building occupants to safely exit a building during a fire. Compartment walls contain a fire and slow its spread.

Passive fire protection systems are intended to protect structural elements from severe damage or collapse during a fire. Examples include spray applied fire resistive materials for structural steel, gypsum board assembles or concrete encasement of structural elements. Typically passive fire protection systems are designed and specified by the project architect with little or no involvement of the structural engineer. However, there is a growing consensus that public safety would be better served if structural engineers played a more active role in designing, specifying and inspecting the passive systems that protect the structures that they design.

Figure 1-2 Spray fireproofing operation – *DeStefano & Chamberlain*

Prescriptive and Performance Methods

The Building Code stipulates minimum requirements for passive fire protection of structural and other building elements. The procedures for evaluating these requirements are "cookbook" in nature and are not particularly difficult for a structural engineer to master – no formal training in fire protection engineering is needed. Designing fire protection based on the Building Code requirements is considered a "prescriptive method."

The prescriptive method is conservative in nature and, if properly implemented, should result in a reasonably fire safe structure. Unfortunately, the passive fire protection of building structures is all too often not in compliance with the Building Code requirements due to a lack of knowledge, interest or effort on the part of the responsible design professional, contractor and inspector.

Furthermore, the prescriptive approach is based on testing of single elements or assemblies in a relatively small standard fire test. Although, historically the prescriptive approach has served us well, the limitations of the test mean that true performance of a structure in a real fire is not predictable.

As an alternative to the prescriptive method, there are analytical methods for calculating the fire endurance of structural elements. These analytical methods are "performance based." A realistic estimate of the quantity of combustible building contents and the corresponding fire load, along with knowledge of how the structure behaves locally and globally under the extreme temperatures of an uncontrolled fire, are used to calculate the amount of passive fire protection needed on the structural and building elements.

Performance based methods may be more rational than the prescriptive method, but they can require considerably more effort and expertise. Currently, a performance based analysis or design is usually performed by consultants with special fire protection engineering expertise, and only when there is an economic incentive to reduce the amount of code mandated fire protection or when the prescriptive method produces an architecturally undesirable result. However, there are techniques that can be adopted by practicing structural engineers and it is the intent of this guide to highlight some of these opportunities.

Since the performance based analysis often results in a reduction in the amount of passive fire protection needed, some have argued that it results in a less fire safe building. It is more accurate to conclude that performance based methods result in fire protection materials being placed where they can yield the most benefit to the fire safety of the building and its occupants. Consequently, there can be more confidence in the adequate performance of a structure subjected to fire.

Teamwork

The design of effective fire protection systems is an effort that requires the participation of the entire design team. The Architect, serving in the role of prime design professional is ultimately responsible for all of a building's fire protection systems, but in fact the responsibility for designing most of the systems falls on the shoulders of consultants.

The Mechanical/Electrical/Plumbing (MEP) engineer is usually responsible for designing the sprinkler systems, standpipe systems, smoke control systems, detection and alarm systems. However, it is common practice for the MEP engineer to delegate the responsibility for sizing sprinkler systems to a sprinkler contractor.

The Architect will design the fire egress systems which include stairways, door hardware and corridors. The Architect will also design and specify passive fire protection materials and systems, sometimes with the assistance of the structural engineer, but more often without.

When performance based methods are required, a specialty Fire Protection Engineer is often engaged to perform the analysis and design.

So where does the structural engineer fit in? Typically, fire protection design is not part of the structural engineer's basic services. If it is included in the scope of services, the structural engineer would assist the Architect in selecting appropriate passive fire protection of structural elements by the prescriptive method. In some cases it may be more appropriate for the Structural Engineer to assume prime responsibility for the design of prescriptive fire protection systems relating to the structural framing.

Simple performance based analysis of passive fire protection systems could be performed by a structural engineer (e.g. single element analysis). To become competent at performance based analysis requires more training or self-study than the prescriptive method.

There are extensive requirements in the Building Code for Special Inspection and testing of spray applied fire resistive materials on steel structures. In some cases, the structural engineer will play a lead role in this inspection and testing.

Whatever the level of involvement of the structural engineer is in designing, specifying and inspecting fire protection systems, it is imperative that appropriate fees be negotiated to compensate for the expertise, effort and risk involved.

Chapter Two – Prescriptive Method

The prescriptive method is the approach whereby the selection of fire resistance is based on a rigid set of requirements contained within the *International Building Code* (IBC). When determining the fire resistance rating of a building structure there are basic characteristics that must be considered:

- Occupancy type or types (single use or mixed use)
- Height and area of the building
- Construction type.

The Building Code defines five general construction classifications for buildings, with sub categories within each. The architect or structural engineer must select one of the construction classifications for a particular building project. The Building Code stipulates maximum building height and floor area for each construction classification and use group (Table 503 of the IBC). The code allows increases to the height and area limits if the building is sprinklered, or has more than 25% of the perimeter accessible to fire trucks. For construction economy, the objective is to select a construction classification that has the lowest fire resistance rating requirements.

The Building Code contains special requirements for buildings with mixed uses and for open parking structures. There are also special provisions for buildings with unlimited area.

Selecting the Construction Classification

The International Building Code (IBC) uses essentially the same approach for determining the construction classification that was developed one hundred years ago.

The maximum size (height and area) of buildings is based on the occupancy and the type of construction. The assumption is that the use influences the combustible content and the number and characteristics of the occupants (e.g. transient occupants in hotels, non-ambulatory patients in hospitals, etc.). The fire load is the combustible content plus the combustible construction. The fire load determines the duration (severity) of the potential fire.

The philosophy has been to provide ample time for occupants to safely escape a burning building, to allow safe access for fire fighters and to prevent the fire from spreading to adjacent buildings. Early classifications were as follows:

- Low hazard uses of residential, educational, institutional and assembly having 0 – 10 psf of fire load.
- Medium hazard uses of mercantile having 10 -20 psf of fire load.
- High hazard uses where fire load is greater than 20 psf.

Industrial uses and storage can change from low hazard to high based on combustible content. Other considerations are assembly buildings should be more restrictive because large crowds of people may panic in a fire. Also consideration needs to be made for people not capable of swift evacuation due to age, illness or physical restraint.

Using the above criteria and experience from fires in one story buildings, allowable floor areas per story were established for different uses. At a later time these areas were permitted to be increased based on the use of automatic sprinklers and accessible perimeter to enhance fire fighting. Area is defined as the floor area within the exterior walls or fire walls, exclusive of vent shafts and courts. Excluding shafts and courts from the measurement typically adds about 3-5% to the permitted area.

Height is the vertical distance from the grade plane to the average height of the highest roof surface in feet. A story is that portion of the building between the upper surface of a floor and the upper surface of the floor or roof next above. Basements need not be included unless the floor above is more than 6 feet above grade plane; or more than 6 feet above the finished ground level for more than 50% of the total building perimeter or more than 12 feet above the finished ground level at any point.

Mezzanines having an area not exceeding one-third of the area of room or space below are not to be considered a story.

The initial types of construction were “fireproof” and “non fireproof”. These were later changed to “fire resistive” for obvious reason. In time these expanded to the following five:

- Fire resistive
- Non-combustible
- Exterior protected ordinary
- Heavy timber
- Wood frame

In current code language these have been translated into:

- Type I, A and B (non-combustible, protected)
- Type II, A and B (non-combustible, protected or unprotected)
- Type III, A and B (non-combustible/combustible, protected or unprotected)
- Type IV, Heavy timber
- Type V, A and B (combustible, protected or unprotected)

Type I and II construction includes structural steel or concrete frame buildings where the structure is composed entirely of non-combustible materials. In some instances, fire retardant treated wood is permitted for roof framing.

Type III construction includes buildings with exterior walls constructed of masonry or other non-combustible materials. The floor and roof framing may be wood.

Type IV construction includes buildings framed with heavy timbers. The exterior walls must be non-combustible. The floor and roof decking must be solid wood planking with no concealed spaces..

Type V construction includes all types of wood frame construction.

Design Example – Based on 2003 IBC

5 stories, 60,000 sf per floor, Business Occupancy, accessible perimeter of the building is 25 feet wide for 50% of the perimeter.

From Table 503, Construction Type II B would permit 4 stories and 23,000 sf per floor.

However, if automatic sprinklers are added, one story can be added and the area can be increased by 200%. The area may also be increased based on the accessible perimeter.

Equation 5-2, for frontage increase, W is accessible width, P is total perimeter, F is accessible perimeter.

$$I_f = 100\ [F/P – 0.25]\ W/30$$

$$I_f = 100\ [500/1000 – 0.25]\ 25/30 = 21\%$$

I_s = 200%, for automatic sprinkler increase

Equation 5-1, for total increase in area

$$A_a = A_{tt} + [A_t\ I_f/100] + [A_t\ I_s/100]$$, where A_t is area from Table 503

$$A_a = 23{,}000 + [23{,}000 \text{ x } 21/100] + [23{,}000 \text{ x } 200/100]$$

$$A_a = 23{,}000 + 4830 + 46{,}000 = 73{,}830 \text{ sf per floor}$$

Therefore, the building is in compliance with the IBC for Construction Type II B.

Determining Fire Ratings

Once the construction classification has been selected, Table 601 of the IBC defines the fire ratings required for each building element, floor construction, roof construction, columns, etc.

There are no fire ratings tabulated for brace elements that resist only wind or seismic lateral loads. This is based on the assumption that it is unlikely that a hurricane or earthquake will occur during fire.

The code does require that beams which brace a column must have the same rating as the column that they are bracing. Similarly, beams that support a wall around a stair must have the same rating as the wall.

Fire ratings are listed as restrained assemblies, unrestrained assemblies and unrestrained beams. Fire rated floors or roofs are assemblies. Fire rated beams that brace a column or support a rated wall are unrestrained beams.

Restrained assemblies require less fire protection than unrestrained assemblies with the same fire rating. In the context of fire rated assemblies, the term “restrained” has a different meaning from that commonly used by structural engineers.

A building fire is often limited to a small area of a building and only heats up the structural framing immediately above the fire. If there is surrounding floor or roof construction that is capable of restraining the thermal expansion of the structure in the vicinity of the fire, the assembly will perform better and is considered to be restrained.

The Underwriters Laboratory (UL) publishes a directory that lists the fire rated assemblies, beams, columns and walls that they have tested. UL is not the only testing laboratory that performs fire tests, but they are the most prolific. There are other laboratories that also list fire test results such as Warnock Hersey, Southwest Research Institute, Intertek Testing Services, Omega Laboratories, etc.

Fire tests are defined in ASTM E119. A full size test specimen is placed in a test furnace and subjected to a fire with a prescribed time-temperature curve. The time period to failure is recorded for the test. Since actual building fire conditions are different from an ASTM E119 test, an assembly with a 2 hour rating will not necessarily survive a real fire for 2 hours. The ASTM E119 test is a good method of rating the relative fire resistance of different building elements, but it is not a good predictor of an element’s actual duration in a real fire.

Each listing in the UL Fire Resistance Directory describes in great detail all of the significant components of the test specimen, such as beam size, type and thickness of fireproofing, type, size and gage of metal deck, thickness of concrete slab and type of concrete aggregate. The building construction must match all of the components of the test specimen for the referenced fire endurance test.

References

1. International Code Council (ICC) – *International Building Code* – 2006

Chapter Three – Performance Based Design

Structural Fire Engineering or demonstrating fire resistance by Performance Based Design (PBD) is essentially the design of structural elements, sub-assemblies or frames to support the applied load at high temperatures during a fire. It is not significantly different to designing for wind or any other load except that the material properties (strength and stiffness) of the structure degrade and thermal expansion can generate additional axial forces, moments and deflections that may need to be addressed by the design.

Establishing the code required Construction Type and the required fire resistance ratings for a structural frame has traditionally been the responsibility of the Architect. It is common practice for the structural engineer to design for all other structural loads. It is the intent of this chapter to provide structural engineers with an introduction to the necessary information to consider the impact of fire on their designs.

Structural Fire Engineering Process

Structural Fire Engineering is generally a 3 stage process, as follows:

1. Definition of a design fire, which can be the standard Time-Temperature curve from ASTM E119 or a credible design fire.
2. Calculation of heat transfer from the design fire to the surrounding structure. The aim of this stage is to establish the temperature of the affected structure as it varies with time.
3. Structural assessment of the load paths and capacity at the calculated structural temperatures.

The complexity with which each stage is assessed can vary from simple hand calculations to complex computer modeling.

Design Fires

There are a number of fire exposures that a structural element can be exposed to:

- A fully developed fire involving all of the contents in the room / compartment / enclosure.
- A localized fire that is prevented from growing significantly because of sprinkler spray, the compartment height or lack of fuel load.
- External flames projecting through windows.

In terms of choosing a suitable design fire, external structures may only be exposed to flaming through specific window openings. In tall or large spaces with well-defined and specific locations for fire load (e.g. atriums, airport terminals, parking garages) a local fuel-bed controlled fire adjacent to a critical piece of structure might be appropriate as the basis of design. In areas of high fire load (e.g. retail, office, residential) with ceiling heights of 10 to14 feet, a fully developed compartment fire is likely to be the most appropriate fire case.

Fully Developed Fires

The fully developed fire provides the worst case fire exposure to a structure. Real fully developed fires are a function of the dimensions of the compartment, the area and height of window openings, the thermal properties of wall/ceiling and floor linings and the type, configuration, and quantity of fuel in the compartment. When levels of fire resistance are derived from an ASTM E119 standard fire test using a furnace with a defined temperature-time curve, these variables are ignored.

The development of a compartment fire can be described by three distinct phases, the pre-flashover fire, the fully-developed fire (or post-flashover fire) and the cooling phase. There is a rapid transition stage called flashover between the pre-flashover and fully developed fire. While still small (during the growth phase) the compartment fire will behave as it would in the open. As it grows the confinement of the compartment begins to influence its behavior. If there is sufficient fuel and ventilation (i.e. window glass breaks) the fire will develop to flashover and its maximum intensity when all combustible surfaces are burning. If the fire is extinguished before flashover or if the fuel or ventilation is insufficient there will only be localized damage. Post-flashover, the whole enclosure and its contents will be devastated. Structural damage and fire spread beyond the compartment of origin is also likely unless the fire is in a fire rated enclosure. Fire resistance is important when elements of structure are subjected to high temperatures for a prolonged period of time. Post-flashover fires provide the worst case scenario.

The ASTM E119 standard fire test which forms the basis for code defined fire resistance ratings, is intended to represent a fully developed fire involving all of the contents in the room. However, it has been widely criticized for its inability to reflect the real fire case. The difference between the standard test Time - Temperature curve and the Time - Temperature curves measured in real compartment fires can be considerable.

Localized Fire

Empirical equations and complex computer software, such as Fire Dynamics Simulator (FDS), exist to calculate local fire cases. They enable the user to calculate flame heights, plume temperatures at various heights, and temperatures along ceiling jets. This data can then be used to calculate the heating effect to and through the structural elements for the duration of the fire.

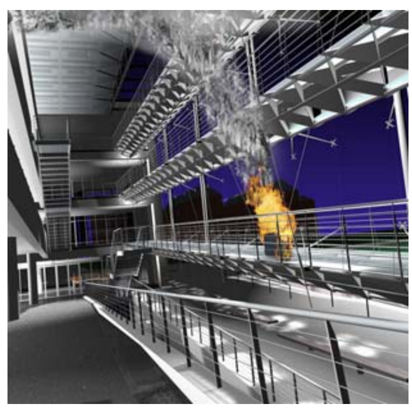

Figure 3-1 localized fire - *Arup*

In order to calculate the flame height the engineer has to make an assumption about the size of the expected fire in British Thermal Units (BTUs) per second. The *Society of Fire Protection Engineers* (SFPE) handbook, the *National*

Fire Protection Association (NFPA) Fire Protection Handbook and NFPA 92 B provide some data in this regard. Fire test data from organizations such as the *National Institute of Standards and Testing* (NIST) is also available on the internet.

External Flaming

Empirical equations also exist to calculate external fire exposures. The calculations consider the different fire exposure experienced by external structural members as compared to the same members in a fire compartment. The calculations account for:

- The fire being within an adjacent compartment
- No heat build-up since the member is outside
- Cooling from surrounding air
- Heating based on flame size and position of member with respect to the façade
- Radiative heat flux from the fire compartment
- Radiative and convective heat flux from the external flames through the windows
- Radiative and convective heat loss from the steelwork to the ambient surroundings
- Size and location of the structural steelwork
- Through-draught conditions if windows are broken on two opposing sides of a fire compartment.

FDS can also be used to calculate temperature changes with time for external flaming. The results provided by the software are sensitive to the input data and therefore fire analysis at this level of detail should only be undertaken by an engineer with a good knowledge of fire dynamics and fire chemistry.

Heat Transfer

Once a temperature-time regime for the space or the vicinity of a structural element has been established the structural temperatures can be calculated using heat transfer equations. Various empirical and analytical equations exist. They consider conduction, convection, and radiation in accordance with the principles of heat transfer.

For more complicated structural sections a finite element heat transfer analysis can be carried out.

Structural Analysis

Structural analysis for fire is similar to analysis for other loads and can be based on single member analysis or a frame approach.

Single element analysis is a fast, effective technique that can bring substantial value to a design as it allows an understanding of the overcapacity or under capacity of the member in the fire limit state. This can be used to formulate adequate fire resistance ratings or to inform a change to the structural design to allow adequate capacity without any added fire protection.

Using single element analysis typically involves comparing the structural load that the member has to carry in the fire condition with the capacity of the member at elevated temperature. Axial force, bending, shear, and buckling, including lateral torsional buckling, all need to be considered as appropriate. Buckling events are dependent on stiffness rather than strength. Stiffness degrades more quickly than strength in some materials therefore buckling can occur at lower temperatures than the yield temperature of the material. The capacity of the associated connections also needs to be examined.

It is not typical to consider thermal expansion in single element calculations, therefore, the forces experienced by the structural element in a fire when restrained by adjacent members in a structural frame are not adequately considered. However, in general, single element calculations can be assumed to be conservative because of alternative load paths that can be present in a real frame during a fire are neglected.

Sub-frame or global analysis provides a more accurate assessment of structural performance in fire because it allows the engineer to assess the forces generated by thermal expansion and thermal bowing. It also permits the inclusion of load transfer paths as different parts of the structural frame become weakened, buckle/deform or lose stiffness at high temperatures. Finite element analysis is required for this type of frame analysis. Some consulting engineering firms offer this type of analysis to check the robustness of a structural frame solution when exposed to fire and if appropriate to reduce fire resistance ratings or remove fireproofing where the code requirements can be shown to be excessive. Almost any design and any fire scenario can be addressed in this way.

An important part of structural fire engineering is defining the applied load assumed to act on the structure during a fire event. The factors applied to imposed and dead load in the fire design are typically reduced from that assumed by the structural engineer for normal design. It is not reasonable for structural members to be designed for a high live load during an extreme event such as a fire. The factors applied to dead and imposed loads in fire resistance calculations given by SEI/ASCE 7 are:

1.2 Dead + 0.5 Live + 0.2 Wind

References

1. FEMA, *World Trade Center Building Performance Study: Data Collection, Preliminary Observations and Recommendations*, FEMA Report 403, 2003.
2. Lamont, S.; Lane, B. , Flint, G. , and Usmani, A. S., *Behavior of structures in fire and real design – a case study,* Journal of fire protection engineering, Volume 16, Number 1, February 2006.

3. Magnusson S.E. and Thelandersson S., *Temperature-time curves of complete Process of fire development-theoretical study of wood fuel fires in enclosed spaces*. Technical Report 65, 1970.

4. NFPA *Fire Protection Handbook.*

5. NFPA 92B: *Standard for Smoke Management Systems in Malls, Atria, and Large Areas*, 2005 Edition.

6. NIST Building and Fire Research Laboratory, http://www.bfrl.nist.gov/

7. SFPE *Handbook of Fire Protection Engineering*, 3rd edition. www.sfpe.org

8. SFPE *Engineering Guide - Fire Exposures to Structural Elements*, 2004

9. SFPE *Engineering Guide for performance Based Fire Protection*, 2nd edition

Chapter Four – Fire Testing

Building code requirements for structural fire resistance are based on laboratory tests conducted in accordance with the ASTM E119 *Standard Test Methods for Fire Tests of Building Construction and Materials.* NFPA 251 and UL 263 are fire endurance tests that are virtually identical to ASTM E119. Since its inception in 1918, the ASTM E119 fire test has required that test specimens be representative of actual building construction. Achieving this requirement in actual practice has been difficult since most available laboratory facilities can only accommodate floor specimens on the order of 15 ft. x 18 ft. plan area and 9 ft. x 11 ft. for walls in a fire test furnace. Even for relatively simple structural systems, realistically simulating the thermal restraint, continuity and redundancy globally present in actual buildings is physically impossible to achieve in a test assembly within the ASTM E 119 fire test furnace.

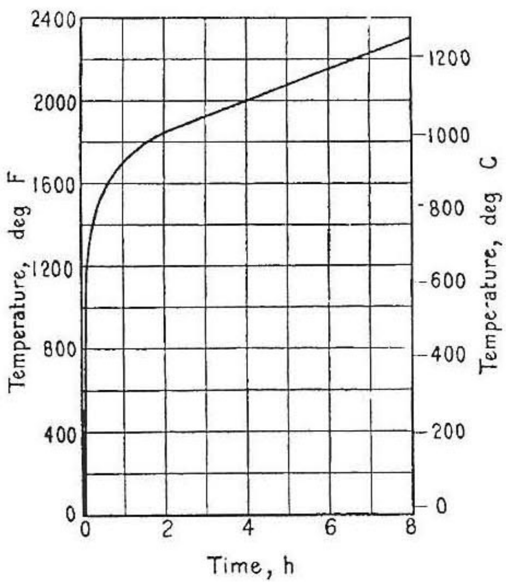

Figure 4-1 Time Temperature Curve - *reprinted with permission from ASTM*

The ASTM E 119 Standard Fire Test was developed as a comparative and not a predictive test. In effect, the standard fire test is used to evaluate the relative performance (fire endurance) of different construction sub-assemblies under controlled laboratory conditions.

For natural fires there are four recognized stages, that of incipient stage, growth stage, burning stage, and a stage of decay. In the ASTM E119 test, the specimen is subjected to heat applied on one side in accordance with a standard time-temperature curve reaching 1000º F in 5 minutes, 1700º F in 1 hour. This represents the most intense burning stage of a real fire, and because the fire in the test apparatus has a constant and controllable fuel source, this permits a consistent baseline for comparison of fire endurance. The test is continued until the specimen fails to meet the criteria for acceptance. The time is recorded to the nearest minute even though ratings are generally published to the nearest hour exceeded.

For floors and roofs, ASTM E119 provides for testing of restrained assemblies and also for unrestrained assemblies with different acceptance criteria. The restrained test leads to two distinct assembly ratings – restrained and unrestrained – by applying different acceptance criteria. The unrestrained test leads to an unrestrained rating with a third set of acceptance criteria.

The ASTM E119 test acceptance criteria includes multiple components:

- Flame passage – no passage of flame or gases hot enough to ignite cotton waste on the unexposed side of a floor, roof, partition or wall.
- Heat transfer – limits are established for the temperature rise on unexposed sides and for structural elements.
- Load support – a superimposed load intended to simulate a maximum loading condition must be maintained.
- Hose stream – wall and partition test specimens are subjected to a hose stream intended to simulate the impact, erosion and cooling effects of a fire hose. No passage of the stream sprayed on the wall is permitted for one half of the resistance period, but not more than one hour.

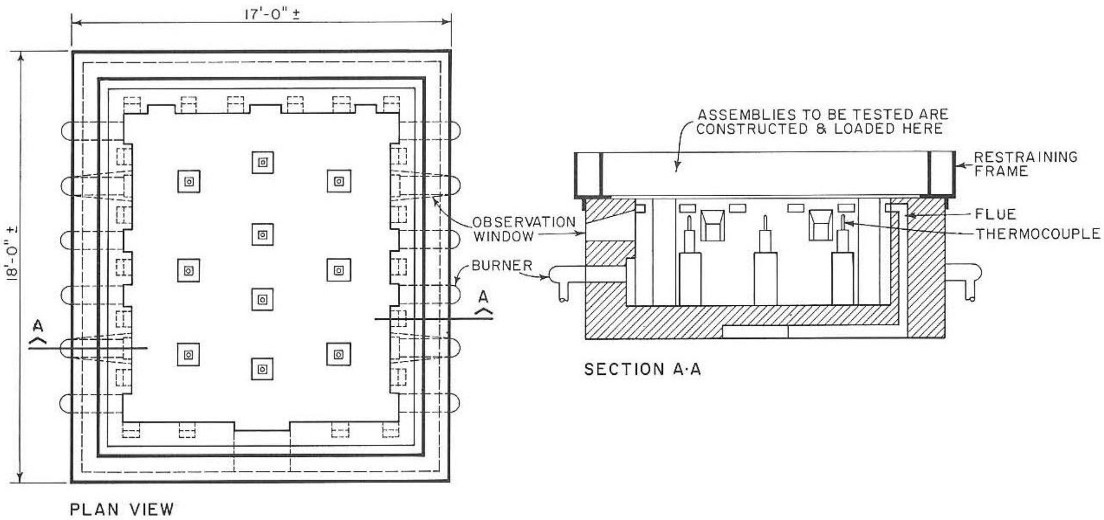

Figure 4-2 Standard fire test - *AISI*

UL Fire Resistance Ratings

In North America, the UL Fire Resistance Directory published annually by Underwriters Laboratories, Inc. is the most widely used compilation of fire resistance ratings. The Design Information Section of this Directory includes useful guidance on the proper design and application of UL's listed ratings.

In the UL test frames, structural connections are rarely included as part of the test assemblies. Beams in fire tests are generally supported on shelf angles with shims driven between the ends of the beams and the test frame to simulate a thermally restrained condition. Although this provides restraint against axial thermal expansion, it does not mimic the rotational restraint found in real structures. Concrete slabs are cast tightly against the test frame, but shrinkage during curing often results in something less than a fully restrained condition. Consequently the support conditions of structural assemblies in a UL test specimen do not accurately model connection behavior, continuity and boundary conditions of a typical floor construction

References

1. ASTM E119 - *Standard Test Method for Fire Tests of Building Construction and Materials*
2. NFPA 251 – *Standard Methods of Tests of Fire Endurance of Building Construction and Materials* - 2006
3. ANSI / Underwriters Laboratory 263 - *Fire Tests of Building Construction and Materials*

Chapter Five – Thermal Restraint

Floor and roof systems are classified as either “thermally restrained” or “thermally unrestrained.” This dual classification was first introduced in 1970 in the ASTM E119 standard. Appendix X3 of the standard contains the following definition:

> *A restrained condition in fire tests, as used in this test method, is one in which expansion at the supports of a load carrying element resulting from the effects of the fire is resisted by forces external to the element. An unrestrained condition is one in which the load carrying element is free to expand and rotate at the supports.*

Table X3.1 of the ASTM E119 standard lists various types of floor and roof construction and classifies each as either thermally restrained or unrestrained. In almost all instances, structural steel construction, cast-in-place concrete and precast concrete construction is considered thermally restrained. Wood frame and timber construction is considered thermally unrestrained. Light-gage cold-formed steel framing is also considered to be thermally unrestrained.

In recent years, in spite of supportive research, fire testing and actual fire experience, the validity of classifying structural steel construction as thermally restrained has been questioned. The International Building Code requires the design professional to designate whether floor and roof systems are thermally restrained or unrestrained. The code requires documentation to be provided to the building official as evidence of a thermally restrained condition. Consequently, the thermally unrestrained classification is the “default” selection for the design professional and building official. The code does not specify what documentation is required to qualify as sufficient evidence of a restrained condition.

Figure 5-1 Restrained structural steel framing sustained large deflections without collapse during a fire test in Cardington, UK - CORUS Steel

It is noteworthy that, although the restrained/unrestrained construction classifications have been in place in the USA and Canada for more than 35 years, other developed countries never adopted them. The restrained/unrestrained classifications are unheard of among fire resistance regulations elsewhere around the globe.

Some of the confusion about “thermal restraint” lies in the traditional meaning of “structural restraint” within the structural engineering profession. The term “restrained” is commonly used to describe the degree of rotational rigidity in structural steel connections. Rigid moment connections are referred to as “fully restrained” and flexible moment connections are referred to as “partially restrained.”

In structural steel construction, the “thermal restraint” developed under fire conditions is a combination of two primary effects:

1. Resistance to axial thermal expansion provided by the surrounding framing and floor slab or roof deck.

2. Resistance to rotation of the ends of the beams and girders. This restraint is influenced by connection stiffness, girder or column stiffness, and interaction of the beams with composite or non-composite components of the floor or roof construction.

TABLE X3.1 Construction Classification, Restrained and Unrestrained

I. Wall bearing:	
Single span and simply supported end spans of multiple bays:[A]	
(*1*) Open-web steel joists or steel beams, supporting concrete slab, precast units, or metal decking	unrestrained
(*2*) Concrete slabs, precast units, or metal decking	unrestrained
Interior spans of multiple bays:	
(*1*) Open-web steel joists, steel beams or metal decking, supporting continuous concrete slab	restrained
(*2*) Open-web steel joists or steel beams, supporting precast units or metal decking	unrestrained
(*3*) Cast-in-place concrete slab systems	restrained
(*4*) Precast concrete where the potential thermal expansion is resisted by adjacent construction[B]	restrained
II. Steel framing:	
(*1*) Steel beams welded, riveted, or bolted to the framing members	restrained
(*2*) All types of cast-in-place floor and roof systems (such as beam-and-slabs, flat slabs, pan joists, and waffle slabs) where the floor or roof system is secured to the framing members	restrained
(*3*) All types of prefabricated floor or roof systems where the structural members are secured to the framing members and the potential thermal expansion of the floor or roof system is resisted by the framing system or the adjoining floor or roof construction[B]	restrained
III. Concrete framing:	
(*1*) Beams securely fastened to the framing members	restrained
(*2*) All types of cast-in-place floor or roof systems (such as beam-and-slabs, flat slabs, pan joists, and waffle slabs) where the floor system is cast with the framing members	restrained
(*3*) Interior and exterior spans of precast systems with cast-in-place joints resulting in restraint equivalent to that which would exist in condition III (*1*)	restrained
(*4*) All types of prefabricated floor or roof systems where the structural members are secured to such systems and the potential thermal expansion of the floor or roof systems is resisted by the framing system or the adjoining floor or roof construction[B]	restrained
IV. Wood construction:	
All types	unrestrained

[A] Floor and roof systems can be considered restrained when they are tied into walls with or without tie beams, the walls being designed and detailed to resist thermal thrust from the floor or roof system.
[B] For example, resistance to potential thermal expansion is considered to be achieved when:
(*1*) Continuous structural concrete topping is used,
(*2*) The space between the ends of precast units or between the ends of units and the vertical face of supports is filled with concrete or mortar, or
(*3*) The space between the ends of precast units and the vertical faces of supports, or between the ends of solid or hollow core slab units does not exceed 0.25 % of the length for normal weight concrete members or 0.1 % of the length for structural lightweight concrete members.

Figure 5-2 ASTM E119 table X3.1 *reprinted with permission from ASTM*

Both modes of restraint occur in steel framed buildings and they both contribute to the fire resistance of a structural steel supported floor or roof system. Indeed, there is strong evidence that, of the two modes, rotational restraint is the more significant. Even minimal rotational restraint provided by simple connections is effective in achieving “thermally restrained” performance. This suggests that calculation (documentation) of the amount of thermal restraint that exists in a structural steel frame building is unnecessary.

Prior to 1993, a table similar to X3.1 was included in the UL Fire Resistance Directory. In 1993 the table was deleted and replaced with a description of the UL test frame stiffness. This information about the test frame stiffness has sometimes been misinterpreted. It has been suggested that a building structure must have stiffness greater than that of the test frame to qualify as thermally restrained. This is an erroneous interpretation.

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Chapter Six – Fire Walls

Fire walls have a very specific meaning in the building code. Not all fire-rated walls are fire walls. Fire walls divide a building into separate portions each of which is treated as a separate building. They are used when a building exceeds the permitted size for its construction classifications or if portions of a building are built of different construction types. The detailed requirements for fire walls are contained in Section 705 of the IBC. The requirement that presents a structural challenge is 705.2:

> *Fire walls shall have sufficient structural stability under fire conditions to allow collapse of construction on either side without collapse of the wall for the duration of time indicated by the required fire-resistance rating.*

Although this requirement seems simple and reasonable, it is often not easy to accomplish, particularly in multi-story buildings. There are three commonly used strategies for achieving the structural stability required by 705.2.

Cantilever Fire Walls

Cantilever fire walls are not connected to or braced by the structural framing on either side and cantilever vertically from the foundation. The wall must be self supporting and there must be a complete break in the structural framing. The framing cannot bear on the fire wall in any way.

Cantilever firewalls are usually constructed of masonry or reinforced concrete. They must be engineered to resist lateral loads stipulated in the code, but not less than 5 psf. Cantilever fire walls are a very efficient solution for a one story structures, but they become increasingly impractical as the building height increases.

Double Fire Walls

Double fire walls consist of two walls built back-to-back. Each wall is rigidly connected to and braced by the structural framing adjacent to it. As with cantilever fire walls, there must be a complete break in the structural framing.

If the structural framing on one side of the fire wall collapses during a fire, the wall attached to the framing will collapse along with it, leaving the second wall standing. Each of the two walls in a double fire wall must have the stipulated fire-resistance rating.

The structural framing may bear on the wall or utilize the wall as a shearwall. Any fire-rated non-combustible wall assembly may be used in a double fire wall.

When an egress corridor must pass through a double fire wall the configuration of the rated doors can be challenging. Each wall is required to have rated doors which swing in the direction of egress travel. This will sometimes require a rated vestibule to prevent the doors from swinging into each other.

Double fire walls are a practical solution for multi-story buildings.

Tied Fire Walls

Tied fire walls are usually built on a column line or between a double row of columns. The structural framing will be continuous and run through the fire wall with the wall tied to the structural framing with flexible anchors.

The framing on each side of the fire wall must be engineered to resist the lateral forces associated with the structure collapsing on the opposite side of the wall during a fire. This usually requires a somewhat robust lateral load resisting system.

A variation on a tied fire wall commonly found in older buildings consists of a load bearing masonry wall with wood joists pocketed into each side. A minimum of 4 inches of solid masonry is required between the ends of the joist pockets and the ends of the joists have a diagonal “fire cut” that prevents the wall from being pried over if the joists collapse during a fire. Intermittent strap anchors connect the masonry wall to the bottom of the joists for lateral support.

Tied fire walls are used when it is not practical to interrupt the structural framing such as in the retrofitting of an existing building structure.

Fire walls may be constructed of any approved non-combustible materials. Concrete unit masonry (CMU) has been the material of choice for fire wall construction. In recent years fire walls constructed of light-gage steel framing and gypsum board have become an acceptable alternative to CMU. There has been some opposition from the fire service to the use of light frame fire walls since these materials lack the toughness to resist falling debris.

Reference

1. NFPA 221 *Standard for High Challenge Fire Walls, Fire Walls, and Fire Barrier Walls* – 2006 Edition

Chapter Seven – Structural Steel

Structural steel is a non-combustible material that has demonstrated reasonably good fire performance when adequately protected. At elevated temperatures, the physical and mechanical properties of steel change. As the temperature of steel increases there is a reduction in strength and stiffness as well as an increase in volume.

Fire Protection Materials

There are a variety of materials and products used to fire protect structural steel framing. Concrete encasement, masonry and plaster have been used to “fireproof” steel for over a century. It is more common today to use gypsum board, spray-applied fire resistive materials (SFRM), intumescent coatings, or mineral fiberboards and mats. Fire-protection materials and systems are designed to delay the temperature rise in structural steel.

Other fire-protection methods for structural steel involve rain screens (sprinklers designed to protect steel members) or filling tubular structures with concrete or water.

Spray-Applied Fire Resistive Materials (SFRM)

Most SFRM either utilize mineral fiber or cementitious materials to insulate steel from the heat of a fire.

Figure 7-1 Application of SFRM – *DeStefano & Chamberlain*

The mineral fiber mixture consists of fibers, binders and water. Mineral fiber fire protection material is spray-applied with the dry mixture fed to a spray nozzle where water is added to the mixture as it is sprayed on the steel. The mineral fiber coating is lightweight, non-combustible, chemically inert and a poor conductor of heat.

Cementitious SFRM are composed of a binder mixed with vermiculite, perlite or polystyrene aggregates. Cementitious SFRM are classified as low density, medium density or high density products. The low density products are most common and usually have a gypsum binder. The medium and high density products often use a Portland cement binder.

The mineral fiber SFRM is less costly than the cementitious SFRM, but does not adhere as well to steel and is easily dislodged. Mineral fiber SFRM and low-density cementitious SFRM are not suitable for wet locations or exposed locations where the fireproofing can be dislodged, such as exposed parking garages.

Intumescent Coatings

An intumescent coating has an appearance of a thick film or paint. When exposed to a fire, it chars, foams, and expands significantly in thickness forming an insulating layer.

Two distinct categories of intumescent coating are manufactured – water or solvent based (also referred to as intumescent paints or thin film coatings), and epoxy based (also referred to as mastics or thick film coatings). Water and solvent based coating are thinner (usually up to 5 mm), and mostly intended for controlled environments inside buildings, although some systems are available for external exposures. Epoxy based coatings are thicker (up to 45 mm), and are mostly used for petro-chemical installations.

Figure 7-2 steel column with intumescent coating during fire test - *Leighs Paint*

In many instances, the intumescent coating is actually a system of multiple coats with different properties and functions. The base coat will be formulated to provide a strong bond to the steel substrate, while the top coat (or the sealing/decorative coat) will be formulated to provide a durable aesthetically appealing finished surface. The intermediate layer of the fire protective intumescent material is usually applied in multiple coats, to achieve the desired protection thickness, allowing sufficient time for each coat to dry before applying the next coat. Depending on the design thickness, the application of intumescent coatings could be a lengthy and costly process.

The high cost of intumescent coatings limits their use to projects where it is important to architecturally expose the structural steel.

During a fire, the intumescent coating will expand up to 50 times its original thickness. For it to be effective there needs to be a space for the coating to expand into. Consequently, intumescent coatings should not be used in tight spaces where there is not sufficient room to allow the coating to expand.

Gypsum Board Products

Type X and Type C gypsum board is used for fire protection. These products have a specially formulated gypsum core that provides a greater fire resistance than regular gypsum board of the same thickness.

Gypsum contains chemically-bound water of crystallization. The fire retarding property of gypsum board is derived primarily from this water content. When the gypsum board is exposed to fire, the water of crystallization is gradually released and evaporated, consuming large amounts of energy in the process and delaying heat transmission through the board. Gypsum board effectively acts as a fire barrier until most of its water content is driven out. Even dehydrated gypsum board acts as a shield against fire flames.

Fibrous Board and Mat Products

Although the mineral fiberboard and similar mat products are usually more expensive than SFRM, they are relatively easy to install as no wet processes are involved. They are often used in retrofit applications and projects where speed and dry process are of importance.

Surface Preparation for SFRM

Figure 7-3 SFRM applied over bare steel – *DeStefano & Chamberlain*

SFRM is intended to be applied over bare, unpainted steel. A light coating of rust will actually improve the adhesion. If the steel has been painted there are special requirements to ensure adequate adhesion of the SFRM. Unless the paint system has been formulated to be compatible with SFRM, additional measures will be required such as applying a bonding agent to the steel or securing metal lath to the steel prior to applying SFRM. The rules for evaluating the adequacy of SFRM bond to painted steel are described in the UL Directory.

Applying SFRM to open web steel joists can be particularly problematic. There tends to be an excessive amount of over-spray and wasted material. It is advisable to tie wire lath to the joist webs prior to applying SFRM although this step will add considerable cost. It is more common to protect open-web steel joist construction with rated ceiling assemblies rather than with SFRM.

Many fire rated floor assemblies require the underside of the metal deck to be spray fireproofed along with the beams. Often galvanized metal deck will have a light film of oil on its surface that needs to be removed by solvent cleaning.

The Prescriptive Method

Qualification fire resistance testing in accordance with ASTM E119 is used extensively to satisfy building code requirements for fire resistance. In order to comply with fire resistance rating requirements, the architect or engineer usually selects suitable fire resistant designs from the UL Directory. Listed designs must be followed in every detail, in order to maintain the fire-resistance rating.

W/D Ratios

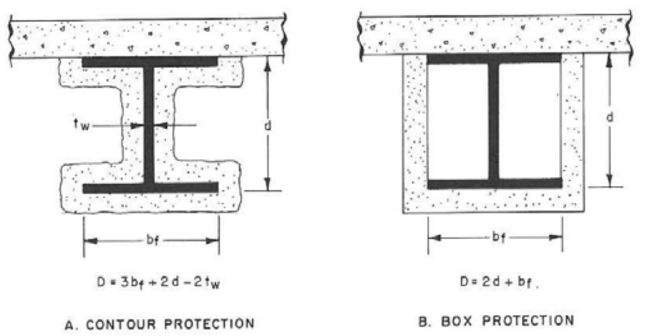

Figure 7-4 Heated perimeter (D) for wide-flange beam - *AISI*

The rate of temperature rise in a structural steel member is a function of its thermal mass and the surface area exposed to heat. Therefore, the factor commonly used in fire resistant design is the W/D ratio, where W is defined as the weight per foot of the steel member in pounds, and D is defined as the inside perimeter of the fire protection in inches. Similar A/P ratios are used in the fire resistant design of tubular column sections, where A is the section area in square inches and P is the section perimeter in inches (identical to D). Values of W/D and A/P ratios for various sections and configurations are tabulated in the AISC Steel Design Guide 19.

The International Building Code and ASCE/SEI/SFPE 29 *Standard Calculation Methods for Structural Fire Protection* have many empirical correlations based on the W/D ratio to allow engineers to calculate the fire resistance of a particular steel section protected with different materials, including concrete, masonry, SFRM and gypsum board.

SFRM Thickness Adjustment

Figure 7-5 thickness testing of SFRM – *DeStefano & Chamberlain*

The UL design will indicate the SFRM thickness that is required for a given fire rating. This thickness is valid only for the beam size that was tested. For instance, UL D739 was based on a W8x28 steel beam. Unless all of the beams on a project are W8x28s, the SFRM thickness will need to be adjusted for each beam size used.

There is a simple formula that is used to calculate the required SFRM thickness based on the W/D or A/P ratio of the steel section.

$$h_2 = \left[\frac{(W_1 / D_1) + 0.6}{(W_2 / D_2) + 0.6} \right] h_1$$

Where h = thickness of SFRM

The project specifications should require the fireproofing contractor to submit W/D calculations for approval along with a schedule of fireproofing thickness for each beam and column size on the project.

Floor and Roof Construction

Fire rated restrained floor and roof assemblies will usually be selected to satisfy the fire resistance requirements for floor and roof construction including beams and joists.

The International Building Code contains fire rating requirements for beams or girders that frame directly into a column. This is intended to ensure that columns remain braced against buckling during a fire. Fire rated floor and roof assemblies are listed under the D series designation of the UL directory.

Structural Steel Columns

IBC Table 719.1(1) prescribes many older column fire rated designs using generic materials, such as concrete, masonry, plaster and gypsum wallboard. Also, IBC Section 720.5.1 contains several equations (with W/D variable) and relevant tables for the calculation of fire resistance of steel columns protected with generic materials. Further, IBC Equation 7-13 allows the adjustment of thickness of proprietary SFRM materials based on the W/D ratio of the column section. ASCE/SEI/SFPE 29 standard contains very similar provisions for steel columns. In addition, the latter provides an equation for the determination of fire resistance of concrete filled tubular steel columns.

Fire resistant steel column designs using proprietary materials, such as SFRM and intumescent coatings, are listed under the X and Y series designation of the UL directory.

Steel Trusses

The inherently large size of truss assemblies does not allow their adequate fire resistance testing in standard furnaces. However, several conservative approaches have been developed over the years for truss fire protection. One common approach is to protect each truss element to the same level as a column of a similar or smaller section size. Another conservative approach, sometimes used for lighter trusses, is to apply proven fire resistant joist designs to heavier trusses. Both approaches are based on the rationale that larger/heavier truss elements would heat up slower than smaller column sections or lighter joists under similar fire exposures.

Individual Protection

IBC section 714.2.1 requires that fire-rated columns, beams, girders, trusses and other structural members “shall be individually protected on all sides for the full length” where the structural element supports:

- more than two floors (or more than one floor and one roof), or
- a loadbearing wall, or
- a non-loadbearing wall that is more than two stories high.

The requirement applies to most columns in multi-story buildings, and effectively prohibits protecting more than one column in a single fire protection enclosure. This individual protection requirement also prohibits the protection of “critical” beams, girders and trusses by fire rated ceilings. However, ceiling protection can be used for regular beam, girder or truss systems supporting one floor or transfer beams, girders and trusses supporting not more than two floors.

Design by Engineering Analysis

For a given heat exposure history (fire scenario), the engineering analysis of a steel structure would involve two major stages. First, heat transfer analysis is conducted to establish the temperature field history in the structure. Second, the structural analysis of the heated steel structure is performed using one the following methodologies:

- Critical temperature approach, the simplest analysis methodology, involves the determination of critical temperatures for various steel elements, and ensuring that these critical temperatures are not exceeded for the required time in the design fire scenario.
- Simple calculation methods are generally ‘hand’ calculation methods, although they are not necessarily simple to use. These calculation methods are based on well-established principles, such as plastic analysis of sections, and they usually are used to analyze one single member at a time. They often involve simplifying assumptions, such as neglect of thermal expansion, temperature-independence and idealization of structural boundary conditions, approximation of second-order effects, and simplified material property models.
- Advanced calculation methods are generally finite element models incorporating geometrical and material nonlinearities, and they are usually used to analyze assemblies of structural components and/or entire building frames. The many assumptions and approximations in advanced calculation models are usually of higher order of refinement than in simple calculation methods, therefore, a higher degree of accuracy is expected. Significant expense is involved in advanced modeling and calculation; therefore, these methods are rarely used for routine design projects. The validity of

Figure 7-6 Cardington fire test - CORUS Steel

the advanced calculation methods was verified in the full scale fire tests performed at Cardington in the UK.

Simple Calculation Models

Simple calculation methods are based on AISC design provisions for the capacity of structural steel and composite members at room temperature, adjusted for the deterioration of the mechanical properties of steel and concrete at elevated temperatures. The provisions cover simple methods for tension, compression and flexural steel members, and also, for composite floor members. The major limitations and simplifications associated with the simple methods of analysis are as following:

> *"The methods of analysis in this section are applicable for the evaluation of performance of individual members at elevated temperature during exposure to fire. The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure"*.

The AISC Steel Design Guide 19 provides a step-by-step procedure for the calculation of flexural capacity of composite floor beams at elevated temperatures.

Influence of Load

The failure temperature of a structural steel member is a function of the load that it is carrying. If a steel member needs about 75% of its ambient strength to support the load then the failure temperature is likely to be about 500 °C (932 °F). If the member is over designed and only needs 40% of the ambient strength then the failure temperature will be higher (~620 °C or 1148 °F). The load at the fire limit state is reduced from that in normal design. Consequently, in localized fires or in situations where a steel member is partially shielded from direct flaming, steel members can resist the applied load at the fire limit state without fireproofing.

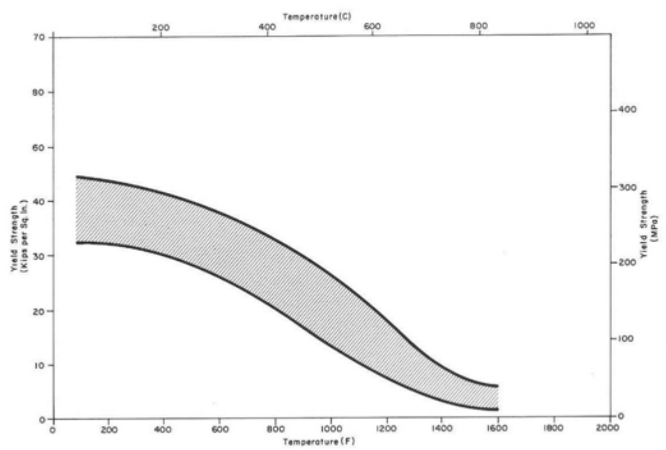

Figure 7-7 Yield strength of A36 steel at elevated temperature - *AISI*

Restraint and Catenary Action

If a steel beam is restrained at its ends by connection to another beam or column then its fire performance is substantially improved by catenary action. Restrained beams can sustain significant deflection, sometimes as much as several feet, with failing. This is because the restrained beam can support the load in axial tension as it hangs from the connections. Therefore, beams in a real building can be expected to perform better than beams in a furnace test.

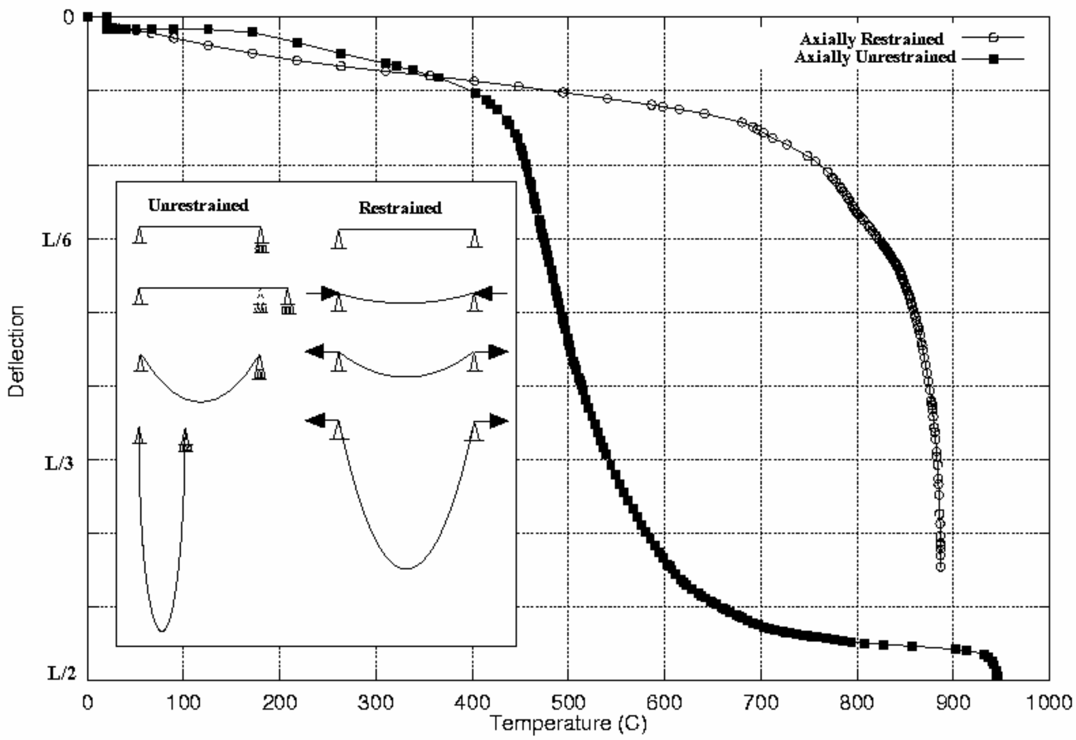

Figure 7-8 Restrained and Unrestrained steel beam failure temperatures - *Lamont and Usmani*

Partially Exposed Steelwork

Standard fire tests on partially exposed steel have shown that structural members not fully exposed to the fire exhibit increased levels of fire resistance. This is related to the W/D concept because parts of the steel are shielded from fire therefore "D" is reduced. The most common methods of achieving partially exposed steel are:

- Web in-filled columns: Normal weight concrete is poured between the flanges of the column. The load carrying capacity of the concrete is ignored in the design of the column but during a fire as the steel weakens the load carried by the flanges is transferred to the concrete providing up to 60 minutes fire resistance.
- Filled hollow sections: Hollow columns can gain enhanced fire resistance by filling them with concrete. During a fire heat flows through the steel to the low conductivity concrete. As the steel loses its yield strength with increasing temperature the load is transferred to the concrete. Adding fiber or bar reinforcement to the concrete can attain enhanced periods of fire resistance
- Water-filled sections: Hollow sections may be filled with water to reduce heating in fire. This method is expensive and infrequently used but is very effective,

although there have been some problems with corrosion and the associated plumbing.

Composite Steel and Concrete Members

Composite steel and concrete members are generally concrete slabs connected to steel beams via shear studs, hollow steel sections filled with reinforced concrete, or steel sections encased in concrete, all of which are designed at ambient temperature, taking into account composite action.

Composite steel frame construction generally has good fire resistance properties as a single member and as a whole frame. The main reason for this good performance is that when the steel member becomes hot and weak, there are alternative paths for the load into the colder concrete slab in a composite floor or the concrete core of a composite column.

The current state of the art for structural fire assessments is to use finite element analysis to calculate the fire resistance of a composite steel frame and design the fire protection layout. The finite element modeling approach allows engineers to quantify the global and local structural performance throughout the duration of a fire and make informed changes to the structural design to improve robustness. This performance based approach will often show that secondary steel infill beams can be left unprotected provided that the girders bracing the columns are protected. This is because alternative load paths exist via catenary action through the composite slab.

Special Inspections

Spray fireproofing is subject to Special Inspections under chapter 17 of the IBC. Testing is required of the fireproofing thickness, bond/adhesion and density. The code requires that one thickness test be performed for every 1,000 square feet of rated floor or roof assembly, and for 25% of the individually rated beams and columns. Previously, one test for every 10,000 square feet was customary. One thickness test consists of averaging several thickness measurements taken on a prescribed pattern.

Figure 7-9 Flutes of metal deck have not been completely filled over beam – *DeStefano & Chamberlain*

In addition to testing, the fireproofing application requires inspection. Some of the common inspection tasks are as follows:

1. Verifying that the flutes of the metal deck have been completely filled above beams.

2. Reviewing cold weather protection methods and temporary heating of the work area.
3. Ensuring there is adequate ventilation to prevent mold from growing in the fireproofing.

It is common for spray fireproofing to be damaged or removed during construction. When hangers are secured to the structural framing to support ceilings, pipes, conduits or ductwork, the tradesmen will often remove portions of any SFRM that is in their way. The Special Inspector needs to verify that damaged or removed fireproofing has been replaced.

Evaluation of Fire Damaged Structures

Following a fire, an engineer is often called upon to evaluate the extent of damage to the structural steel framing and the extent of remedial work needed to put the structure back in service.

Figure 7-10 Fire damaged steel joists - *DeStefano & Chamberlain*

Structural steel is typically manufactured from mild steel with a low to moderate carbon content and relatively low alloy content. Unlike a high-carbon steel, the mechanical properties of mild steel are somewhat insensitive to heat treatment. Since structural steel is typically in a fully annealed state, once the steel has cooled down following a fire its structural properties are similar to those prior to the fire. This can be verified by testing coupon samples cut from beam or column webs.

The same cannot be said for high-strength bolts and nuts. A325 and A490 fasteners are manufactured from high-carbon alloy steels. A490 bolts are quenched and tempered. These fasteners can lose significant strength after being subjected to high temperatures. It is often prudent to replace high-strength bolts and nuts following a fire.

Often the biggest challenge of repairing a fire damaged structure is straightening the steel members. It is common for steel beams and girders to deflect or buckle laterally due to thermal expansion and diminished mechanical properties (strength and stiffness) at elevated temperatures. Members with slight deformations can be straightened with heat methods and jacking, but severely deformed members often require replacement.

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Chapter Eight - Concrete

Concrete construction is naturally fire resistant but, contrary to popular belief, it is not fireproof. Reinforced concrete will sustain damage when exposed to flames and intense heat and will loose strength as its surface spalls and the reinforcing steel heats up.

You will not find many cast-in-place concrete assemblies listed in the UL Directory. Since conducting an ASTM E119 fire test is a fairly costly undertaking, tests are generally only performed on assemblies containing proprietary products where there is a financial advantage for the manufacturer. Cast-in-place concrete, being a somewhat common material produced by numerous small plants, has not been subjected to many commercial ASTM E119 fire tests. Consequently, Architects and Engineers must rely on prescriptive fire resistance tables contained within the IBC for determining the fire resistance of cast-in-place concrete elements and assemblies.

The factors that contribute most to the fire resistance of a concrete element is its thickness, the cover on the reinforcing steel and the aggregate type.

Concrete containing lightweight aggregate has greater fire resistance than concrete containing normal weight aggregate. Normal weight aggregates are classified as either carbonate or siliceous. Limestone (calcium carbonate) and dolomite (magnesium carbonate) are considered carbonate aggregate. Practically all other stone aggregates are silica based and are classified as siliceous. Concrete containing carbonate aggregate has greater fire resistance than concrete containing siliceous aggregate.

Fire Resistance of Concrete Construction

The most common method of designing concrete structures for fire resistance is to ascertain the required thickness and reinforcing steel cover for the required fire resistance rating from tables in Chapter 7 of the IBC. Tabulated data for cover and thickness of concrete elements are also given in ACI 216R-89 and in ASCE/SEI/SFPE 29. However, there are a number of analytical methods to calculate the fire resistance of concrete members based on their capacity in fire.

Spalling

Spalling of surface material from concrete sections is common during a fire but it is a complex process and not well understood. There are various kinds of spalling (aggregate splitting, explosive, surface, corner separation, sloughing off and cooling spalling). The most detrimental to a structural member in fire is explosive spalling. Explosive spalling can occur at temperatures as low as 100 °C (212 °F) and in some cases can fully expose the reinforcement to the full heat of a fire. Explosive spalling is thought to be primarily associated with evaporation of water in the concrete pores and the associated build up of pore pressures. However, restraint to thermal expansion, the heating rate of the fire, the strength of the concrete, and the dimensions of the section also contribute. High strength concrete with added silica fume is particularly susceptible to spalling.

The risk of explosive spalling can be reduced by the choice of aggregate, by limiting the concrete strength, by reducing the moisture content where possible, and by the addition of monofilament polypropylene fibers. The fibers melt at low temperatures generating a larger array of pores in the concrete matrix, which has been shown to significantly reduce explosive spalling by relieving pore pressures.

Material Properties

ACI gives thermal and mechanical properties for reinforcement, prestressing steel, and concrete at elevated temperature. These can be used for detailed analysis to determine the capacity of single concrete elements at elevated temperatures. Again a three-step process (design fire, heat transfer, capacity calculation) is required for concrete.

Heat Transfer

When calculating heat transfer, there are no simple equations that can be used because the conductivity of concrete is low and concrete sections are thermally thick. Consequently, either predefined nomograms drawn for particular sections in response to the standard fire can be used, or a finite element heat transfer analysis is required to calculate the temperature gradient through the depth and/or along the length of a concrete section.

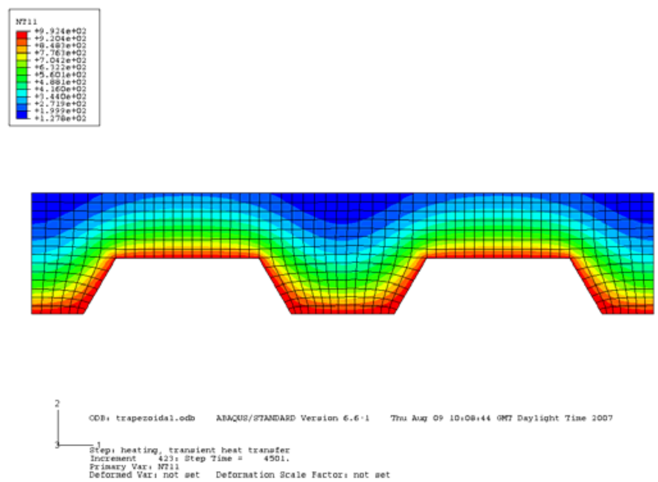


Figure 8-1 2D finite element analysis of concrete slab - *Arup*

Structural Calculations

ACI 216R-89 presents analytical methods for determining the fire resistance of concrete members. The document presents temperature nomograms for slabs and beams in the standard fire and equations for calculating the fire resistance of simply supported slabs and beams, continuous unrestrained flexural members and those restrained against thermal expansion.

Continuous unrestrained members typically have longer fire endurance than simply supported members because they can redistribute moments. The redistribution of moment through the depth of a continuous member can result in failure of the negative moment reinforcement over the supports. However, the ACI warn against increasing this reinforcement as this could result in compression failure of the member, which is not desirable. Limitations are therefore placed on the negative reinforcement to ensure flexural failure in design. Equations to calculate the flexural strength at any point are given.

Equations are also given to account for the thermal thrust force that will occur in the axis of members heated from below and restrained from thermal expansion. The effect of the thermal thrust is the same as prestressing a concrete beam or slab. Therefore, the line of thrust can be considered a fictitious line of reinforcement in flexural design. To calculate the thrust moment, the mid-span deflection must be calculated.

500°C (932 °F) Isotherm Method

The most commonly applied design tool for concrete members, in a compartment with a post-flashover fire, is the 500°C (932 °F) isotherm method or reduced cross-section method.

It is assumed that any concrete at a temperature greater than 500°C (932 °F) will not be able to support load in fire. Any concrete at a temperature less than 500°C (932 °F) can be assumed to have its full ambient design strength. These simple rules and the reduced strength of reinforcement are used to calculate the residual capacity of the section in response to fire. If this residual capacity is sufficient to carry the loads, factored for the fire limit state, the section is deemed to be acceptable.

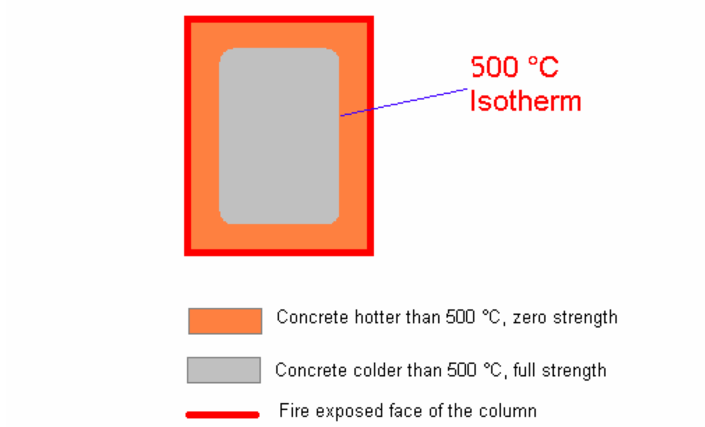


Figure 8-2 500 degree C Isotherm Method - *Arup*

Prestressed Concrete

The performance of prestressed concrete beams and slabs in fire is complicated by the fact that pre-stressed tendons lose strength faster than ordinary reinforcement at elevated temperature and thermal expansion may reduce prestress. The Prestressed Concrete Institute (PCI) has produced guidance on the design of prestressed concrete for fire, which includes analytical expressions to calculate fire resistance. The expressions are based on single element analysis and standard fire testing.

Advanced Analysis of Concrete Structures

Calculation of the whole frame response of concrete frames to fire is possible using finite element analysis. However, none of the computer software that currently exist and are used in fire design explicitly calculate spalling. Therefore the thickness of spalled material must be assumed and generally removed at the start of a structural or heat transfer analysis.

As for any frame analysis in fire, the software must model nonlinear behavior of the structural elements and material properties, as well as include full degradation of material properties with temperature. An accurate representation of thermal gradients varying with time is particularly important for concrete elements as a consequence of concrete's poor conductivity.

Calculating the whole frame response of concrete frames to fire is rarely carried out in design. The main reason for this is the lack of full-scale test data to validate the use of finite element methods with concrete materials.

Summary

Guidance in ACI standards and other international design documents enable designers to calculate the fire resistance of concrete analytically. However, as a result of the simplicity of the tabulated prescriptive guidance, the complexity of spalling and the fact that there is no additional material (i.e. fire protection) required to achieve fire resistance then analytical expressions are rarely used in design. Performance based design solutions can be useful in demonstrating fire resistance if the concrete has been poured to the wrong thickness or an existing building is being upgraded and does not quite meet the dimensions required by the most recent code.

References

1. ACI 216R-89 *Guide for Determining the Fire Endurance of Concrete Elements.*
2. ACI / TMS 216.1-07 *Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies,* 2007.
3. ASCE/ SEI/ SFPE 29, ASCE/SFPE *Standard Calculation Methods for Structural Fire Protection*, 2005.
4. Concrete Reinforcing Steel Institute (CRSI). *Reinforced Concrete Fire Endurance*, 1980.
5. Gustaferro A.H. and Martin L.D., *Design for Fire Resistance of Precast, Prestressed Concrete*, Prestressed Concrete Institute (PCI), 2nd Edition, 1989.

Fire Protection of FRP-Strengthened Concrete Structures

Fiber reinforced polymers (FRP) are composite materials that were originally developed in the 1940's for use in the aerospace industry. The first building structure application was in Europe and Japan in the 1980's. Eventually FRP materials found its way to North America and are now widely used in the strengthening of concrete and masonry structures.

In general, FRP composites consist of two kinds of materials: strong fibers and a resin matrix. There are several types of fibers but the most common are carbon, glass and aramid fibers. Fibers are woven into a fabric that is several times stronger than steel. The most commonly used resin matrix is epoxy that provides adhesion to the base concrete or masonry surfaces. FRP fabric can be fabricated in unidirectional or multidirectional sheets, which should be specified depending on whether strengthening is required in one or two directions.

Figure 8-3 FRP unidirectional strengthening of concrete joists - *Degenkolb*

The installation process typically consists of patching the base surface so that it is smooth and clean of debris. Next, epoxy is applied to the substrate and the fiber fabric is impregnated with epoxy and applied directly. Curing time varies but it could take up to a few days for the system to completely cure.

Because of their high strength, lightweight and corrosion resistance characteristics, FRP strengthening has been widely used on bridges, parking garages, concrete slabs, beams and columns. Although FRP is normally more expensive than a similar solution utilizing steel plates and concrete; its ease of installation and thin profile provide benefits that can make it an excellent alternative.

The traditional application of FRP in building structures includes seismic strengthening of existing concrete slabs, beams, walls and columns to increase its strength and/or stiffness. FRP strengthening can also increase the gravity-load carrying capacity due to change of use, construction mistakes, damage repair and corrosion protection.

Fire Behavior of FRP-Strengthened Structures

One of the limitations of FRP strengthening systems is the low temperature endurance of epoxies and fibers. The mechanical properties of fiber and resins tend to degrade very quickly after these materials reach their glass transition temperature, *Tg*. The glass transition temperature is defined as the temperature range's average at which the epoxies change from a solid state to a softer plastic state. This temperature *Tg* ranges from approximately 140 F to 180 F and is dependent on the materials used by each manufacturer. Each manufacturer determines the maximum service temperature.

Under high temperatures, the fibers and epoxy can produce toxic smoke, spread flames and ignite. Direct fire can cause the FRP materials to delaminate, suffer melting or charring. At this point both the bond with the substrate and the structural properties of FRP materials have been compromised. Recent research indicates that the rate of strength degradation under high temperatures is much greater for FRP materials than it is for concrete and mild steel reinforcement. For example, at a temperature of 200 F, concrete will lose approximately 5% of its initial strength while FRP and steel will have lost approximately 15%. At 400 F, concrete will lose 10% of its initial strength, steel 25% and FRP between 60% and 75% depending on the type of fiber. In general, glass FRP tends to have a faster degradation than carbon FRP.

Design Considerations

For seismic applications the loss of strength is not very significant since the probability of having the seismic design event and a fire that can reach the critical temperature is small. However, for gravity-load strengthening, this strength loss can pose a challenge that needs to be addressed by the structural engineer.

There are two main approaches to dealing with fire resistance design of FRP strengthening. One approach is to provide a fire resistance barrier or insulation that protects the FRP. This can be done with the application of layers of gypsum board. This type of protection can be very expensive.

Spray applied fireproofing or intumescent coatings have been deemed ineffective when applied to FRP, as these systems do not prevent the FRP materials from reaching the critical temperature.

Manufacturers are actively researching new ways to protect the FRP strengthening systems from elevated temperatures under fires. One of the manufacturing companies has developed a multi-layer insulation system that consists of a primer, a dash coat layer and additional layers of proprietary fire resistive material to build up a minimum average thickness of 1-5/8” or more. Concrete assemblies tested by Canada’s National Research Council have determined that this proprietary system can provide a fire rating of up to 4-hour for vertical and overhead assemblies. However, cost and appearance can be prohibitive in some applications. A different type of coating can provide a Class 1 (Class A) flame and smoke rating at a much lower cost.

Considering that the primary concern during a fire is life-safety and egress, ACI 440 recommends evaluating the nominal strength capacity of the original structure to determine if it exceeds the capacity required for the services loads for the required duration of the fire. In this approach, the following limit is suggested:

$$(R_{n\theta})existing \geq 1.0S_{DL} + 1.0\ S_{LL}$$ where:

$(R_{n\theta})existing$ = nominal strength of the original (*unstrengthened*) structural member computed in accordance with ACI 216R
S_{DL} = dead loads acting on the *strengthened* member
S_{LL} = live loads acting on the *strengthened* member

Essentially, this criterion permits for the *unstrengthened* structure to be evaluated assuming that the loads acting on the member during a fire event can be assumed to be unfactored loads. This is permissible since the probability that the structure would collapse under typical actual loads is low.

The nominal strength capacity of the original member should be calculated in accordance with the concepts of the American Concrete Institute's Guide for Determining the Fire Endurance of Concrete Elements (ACI 216R).

References

1. ACI 440.2R-02: *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*, 2002.

2. ACI 216R-89 *Guide for Determining the Fire Endurance of Concrete Elements*.

Chapter Nine – Masonry

As with concrete construction, there are very few masonry assemblies listed in the UL Directory. There are tables contained within Chapter 7 of the IBC that list fire resistance ratings for masonry of various thicknesses and composition. When hollow masonry units are used they are converted to an equivalent solid thickness for purposes of establishing fire resistance rating.

Fire Resistance of Masonry

The design of masonry walls is almost always on the basis of experimentally determined fire resistance levels for walls. The only guidance provided on fire resistance is typically a minimum thickness to give a stated fire resistance period ranging from 30 minutes up to 4 hours. This is not unreasonable since fire tests of masonry walls have consistently shown that the fire endurance is governed by heat transmission (insulation criteria) rather than by structural failure.

Figure 9-1 Masonry bearing walls sustained no significant damage in fire - *DeStefano & Chamberlain*

Some codes give thicker dimensions for load bearing walls than non-load bearing walls, but this may not be justified by comparison with standard fire test data. ASTM E119 requires a load to be applied to load-bearing walls however, although no load is applied to a non-load-bearing panel, the edges of the wall are restrained imposing a significant thermal load as the wall tries to expand. This thermally induced load can be higher than the imposed load in a loaded test.

The restraint provided to a wall has a significant impact on its fire performance. A cantilevered wall will tend to bow outwards, away from the fire. A wall pinned at the top and bottom will tend to bow towards the fire. Walls built into the structure with some continuity at top and bottom will tend to bow less than the pinned case and therefore have more fire resistance because deflections are reduced.

Reinforced concrete masonry walls generally exhibit good fire performance because the reinforcing steel is in the center of the wall and therefore insulated from fire. The reinforcing steel limits cracking of the concrete masonry and resists tensile forces as the wall bends towards the fire.

Masonry walls and parapets are often susceptible to lateral stability problems in fires, especially when “pushed” by thermal expansion of the structural framing. Proper detailing of the attachment of the structural framing to masonry walls is crucial to prevent the destabilization of masonry walls in a fire. For instance, it has been common practice for over a century to provide a diagonal “fire cut” on the ends of wood joists and beams that are pocketed into masonry bearing walls. The fire cut prevents the joists from prying the masonry wall over if the wood joists fail during a fire.

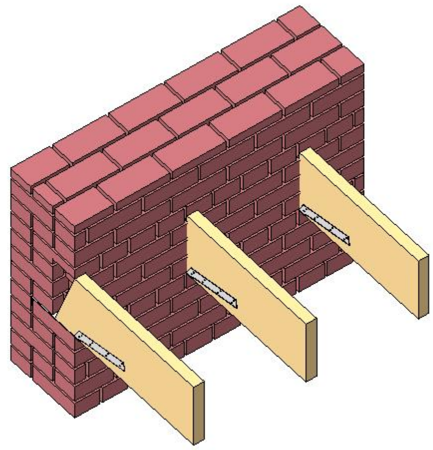


Figure 9-2 "Fire cut" on wood joists bearing on masonry wall - *DeStefano & Chamberlain*

References

1. ACI / TMS 216.1-07 *Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies*, 2007.
2. BIA Technical Note 16 *Fire Resistance of Brick Masonry*, Brick Industry Association, 2008.
3. International Code Council (ICC) *International Building Code* – 2006.
4. NCMA TEK 7-1B *Fire Resistance Ratings of Concrete Masonry Assemblies*, National Concrete Masonry Association, 2008.

Chapter Ten – Wood

Wood is a combustible material which contributes fuel to a fire. Unprotected light wood frame construction and prefabricated wood truss construction in particular has poor fire resistance. Wood framing is usually protected with one or more layers of gypsum board to achieve a fire rating.

Unlike light wood framing, heavy timber construction has demonstrated excellent fire performance.

Fire Resistance of Light Frame Wood Construction

Figure 10-1 Fire damaged light wood framing - *DeStefano & Chamberlain*

The fire resistance of light frame construction is difficult to predict therefore tabulated data and empirical correlations based on standard testing should be used. ASCE/SEI/SFPE 29 presents values of fire resistance for various components of light frame wood construction. The additive method of calculation is proposed, which simply means adding together the fire resistance times assigned to each component as given in the code. For a stud wall for example the fire resistance time is equal to the total of all of the membrane elements on the fire exposed face plus the fire resistance time of the stud framing elements but the membrane on the unexposed face is ignored in the calculation.

Fire Retardant Treatment (FRT)

Wood can be pressure treated with fire retardant chemicals which allow it to be used for certain applications in non-combustible buildings. The FRT chemical formulations are proprietary and vary with the manufacturer.

The most common FRT products are only suitable for interior applications since the chemicals are water soluble and will leach out of the wood if exposed to the weather. There are exterior grade FRT products available that are suitable for wet environments.

Early FRT products were hygroscopic compounds that often caused accelerated corrosion of ferrous metal fasteners, even in a dry environment. These products were replaced in the 1980s with a new generation of low-hygroscopic compounds. Unfortunately, some of the low-hygroscopic FRT products caused degradation of plywood roof sheathing when subjected to high temperature and humidity. Current FRT products have less susceptibility to wood degradation, but the problem has not been entirely eliminated.

Fire Resistance of Heavy Timber

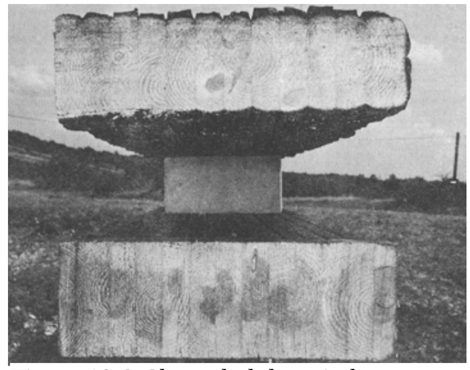


Figure 10-2 Charred glulam timber following fire test - *AF&PA*

The burning behavior of wood is that of a charring material that deposits residue as it burns. This char residue insulates the timber from the heat of a fire and helps prevent further pyrolysis; it does, however, result in a loss of section.

A mechanics based design procedure for calculating the fire endurance of heavy timber is contained in Chapter 16 of the National Design Specification for Wood Construction (NDS). The char rate of timber in a fire is assumed to be the same for all species of wood. After a fire endurance of 1 hour, the effective char layer thickness is assumed to be 1.8 inches. After 2 hours, the assumed effective char layer thickness is 3.2 inches. The char layer is assumed to have no reliable strength or stiffness, so the section properties of the remaining timber section inside the char layer are calculated.

The strength of the timber is calculated based on the reduced section properties. Ultimate strength values are used in the calculation. The published allowable stresses are increased by an adjustment factor "K" to determine the ultimate strength. For flexural strength, the value of "K" is 2.85.

For calculating fire endurance, only dead and live loads are considered since there is a very low probability of an extreme wind or seismic event occurring simultaneous with a fire.

For heavy timber members, ASCE/SEI/SFPE 29 provides analytical methods for calculating the fire resistance time (t) of beams and columns. The equations typically include load factor (z), initial dimensions of the section (b,d), and a constant (γ) with the units min/mm or min/in.

An example for beams exposed to fire on four sides is given in Equation 10-1.

$$t = \gamma z b\left[4 - 2\left(b/d\right)\right] \qquad \textit{(equation 10-1)}$$

Timber Connections in Fire

Where connections are traditional wood joints, the same charring rates as for the main member can be adopted and its capacity in shear can be checked as it would be in ambient design.

Figure 10-3 Timber frame structures exhibit good fire performance - *DeStefano & Chamberlain*

The collapse of traditional wood connections in fire scenarios is preceded by distinctive "cracking" and hissing noises due to the stresses in the wood. Visible deflections also become apparent before collapse, providing the fire services with warning that collapse is imminent.

Timber connections involving steel connectors such as bolts or steel plates mean that the traditional bending and compressive forces in the wood joints have been largely replaced now by localized shear forces surrounding the steel connectors. Since steel is a good conductor of heat, the connections result in elevated temperatures in the wood adjacent to the steel hardware

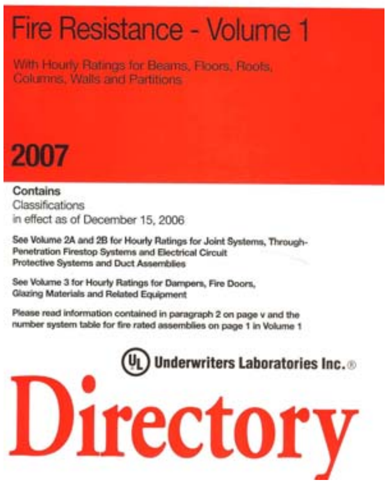
Shear failure occurs when the lignin within the wood section becomes heated and begins to act like a super-cooled liquid. This change occurs at relatively low temperatures ~120 °C (248 °F) and causes the shear strength of the wood to change. This shear failure poses a catastrophic collapse mechanism during a fire scenario because no prior warning of collapse is given under these circumstances. Consequently, metal connectors should be protected. Intumescent coatings are an effective method of protecting exposed steel gusset plates and connection hardware.

References

1. ANSI / AF&PA NDS *National Design Specification for Wood Construction*, American Forest & Paper Association – American Wood Council, 2005.
2. AF&PA Technical Report 10 – *Calculating the Fire Resistance of Exposed Wood Members*, 2003.
3. AF&PA Design for Code Acceptance 3 – Fire Rated Wood Floor and Wall Assemblies, 2007.
4. ASCE/ SEI/ SFPE 29, *Standard Calculation Methods for Structural Fire Protection*, 2005.
5. Gypsum Association, *Fire Resistance Design Manual*, 18th edition.

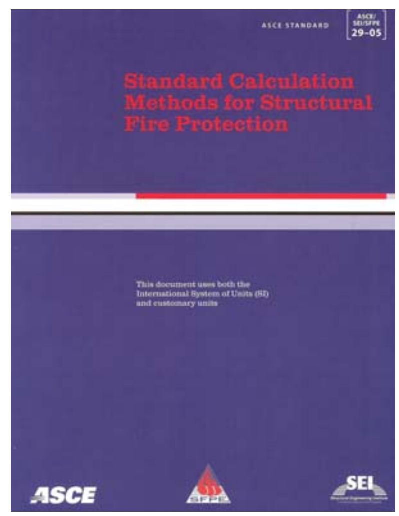
Appendix A – Resources

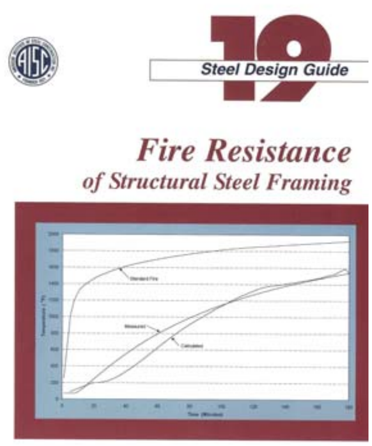
The following documents and publications are useful resources for Structural Engineers engaged in the design of passive fire protection systems.



The *Underwriters Laboratory Fire Resistance Directory* catalogs building elements and assemblies that have been subjected to fire testing by UL. It includes fire test results for beams, floors, roofs, columns, walls and partitions. While not the only source of fire test results, it is certainly the most comprehensive.

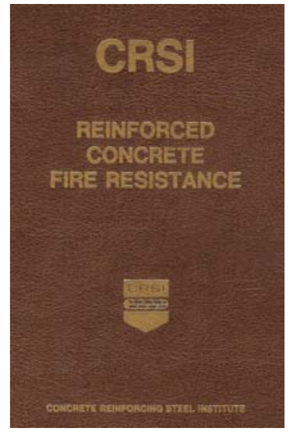
ASCE/SEI/SFPE 29-05 Standard Calculation Method for Structural Fire Protection is a joint standard of the American Society of Civil Engineers, the Structural Engineering Institute and the Society of Fire Protection Engineers. It contains analytical methods for calculating the fire endurance of concrete, structural steel, masonry and wood structural elements.

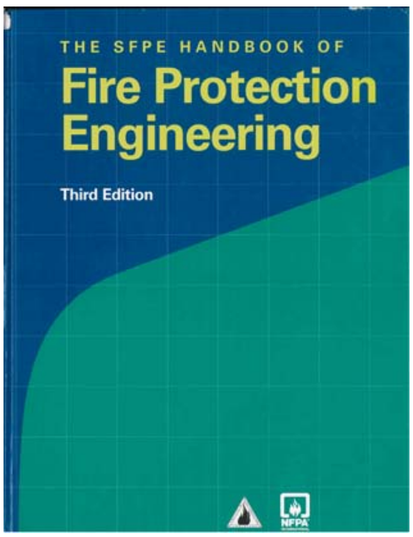




AISC Steel Design Guide 19 – Fire Resistance of Structural Steel Framing is a useful design aide for selecting passive fire protection of structural steel by the prescriptive method. It contains easy to understand explanations of fire protection systems as well as useful charts and tables for determining the required thickness of fireproofing.

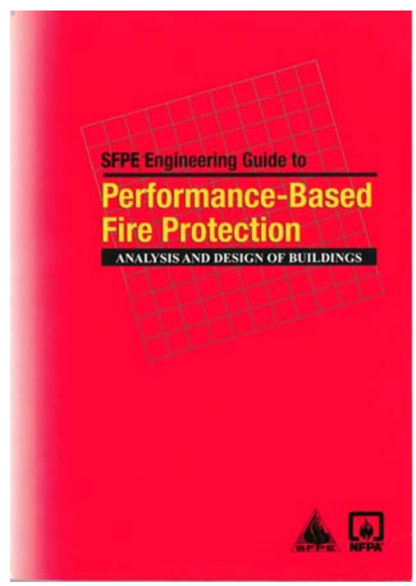
CRSI – Reinforced Concrete Fire Endurance provides a detailed explanation of the fire performance of reinforced concrete structures. Published in 1980, the document contains some outdated Building Code references but it is still a useful document.

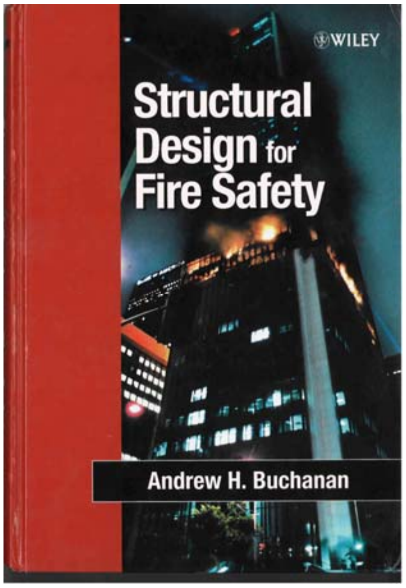




The SFPE handbook of Fire Protection Engineering can be found on the desk of almost every fire protection engineer in the world. It provides detailed information on of all aspects of fire protection engineering including egress and people movement, detection and alarm, smoke control systems, sprinkler design, fire test methods, properties of materials at high temperatures and computer modeling of fire and smoke. It contains two chapters on the subject of fire resistance but a number of chapters which provide test data and guidance for defining design fires or conducting a heat transfer analysis. It is a useful guide for engineers who want to learn more about the principles of fire engineering.

The SFPE Engineering Guide to Performance Based Fire Protection – Analysis and Design of Buildings outlines an approach or process for a performance based design and assessment of building fire safety. It focuses on the application of scientific and engineering principles to design, identifies parameters that should be considered in the design, helps the user set performance objectives and provides information that an authority having jurisdiction can use to assess a performance based solution. It is not specifically for fire resistance but a general approach to any performance based solution.





Structural Design for Fire Safety provides a thorough review of many of the issues relating to the fire resistance of structural elements. It is a useful guide for engineers who want to learn more about performance based design for fire resistance and covers in more detail many of the concepts introduced in this Guide.

Technical Report 10 – Calculating the Fire Resistance of Exposed Wood Members – American Forest and Paper Association. This technical report provides a detailed description of the mechanics based design procedure for calculating the fire endurance of heavy timbers construction.

Acknowledgement

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Disclaimer

This document is intended to provide information in regard to the engineering of structural fire protection. It is not to be regarded as providing opinion or advice for any individual project. This document is distributed with the understanding that CASE is not engaged in rendering professional services. If professional advice is desired, the services of a competent professional should be sought.