

# A Overview of Fire Protection in Buildings

## A.1. Introduction

This appendix presents background information on the fire and life safety aspects of buildings for the interested reader. This review of fire behavior outlines burning characteristics of materials as well as the effect of building characteristics on the temperatures experienced. The description of the effect of fire exposure on steel and concrete structural members is intended to improve understanding of how these structural members respond when heated and also what measures are commonly used to limit temperature rise in structural members. Finally, a brief discussion on evacuation behavior in high-rise buildings is included to provide some context to the comments made in the report concerning the design of the means of egress and the evacuation process in WTC 1 and WTC 2.

## A.2 Fire Behavior

Important aspects of fire behavior in the affected buildings involves the following issues:

- burning behavior of materials, including mass loss and energy release rates
- stages of fire development
- behavior of fully developed fires, including the role of ventilation, temperature development, and duration

### A.2.1 Burning Behavior of Materials

Once a material is ignited, a fire spreads across the fuel object until it becomes fully involved. The spread at which flame travels over the surface of the material is dependent on the fuel composition, orientation, surface to mass ratio, incident heat, and air supply. Given sufficient air, the energy released from a fire is dictated by the incident heat on the fuel and the fuel characteristics, most notably the heat of combustion and latent heat of vaporization. The relationship of these parameters to the energy release rate is given by:

$$\dot{Q}'' = \frac{\dot{q}''}{L_v} \Delta H_c \quad (\text{A-1})$$

where:

$\dot{Q}''$  = energy release rate per unit surface area of fuel

$\dot{q}''$  = incident heat per unit surface area of fuel (i.e., heat flux)

$L_v$  = latent heat of vaporization

$\Delta H_c$  = heat of combustion

The effective heat of combustion for a mixture of wood and plastics is on the order of 16 kJ/g. For fully developed fires, the radiant heat flux is approximately 150 to 200 kW/m<sup>2</sup>. The latent heat of vaporization for a range of wood and plastics is 5 to 8 kJ/g. Thus, the mass burning rate per unit surface area in typical office building fires ranges from 20 to 40 g/m<sup>2</sup>-s and the associated energy release rate per unit surface area ranges from 320 to 640 kW/m<sup>2</sup>.

In typical fires, as the fire grows in size, the energy release rate increases to a peak value as depicted in Figure A-1. The increase in the heat release rate with time depends on the fuel characteristics, incident heat, and available air supply. Sample curves for alternate materials, described in the fire protection literature as “slow,” “medium,” and “fast” growth rate fires, are illustrated in Figure A-2.

At some point, the heat release rate of the fire will become limited by either the amount of fuel or the amount of oxygen that is available; this is referred to as the peak heat release rate. Peak heat release rate data can be obtained through experimental testing and is available for many types of materials and fuels. Table A.1 includes a list of selected common items and their associated peak heat release rates.

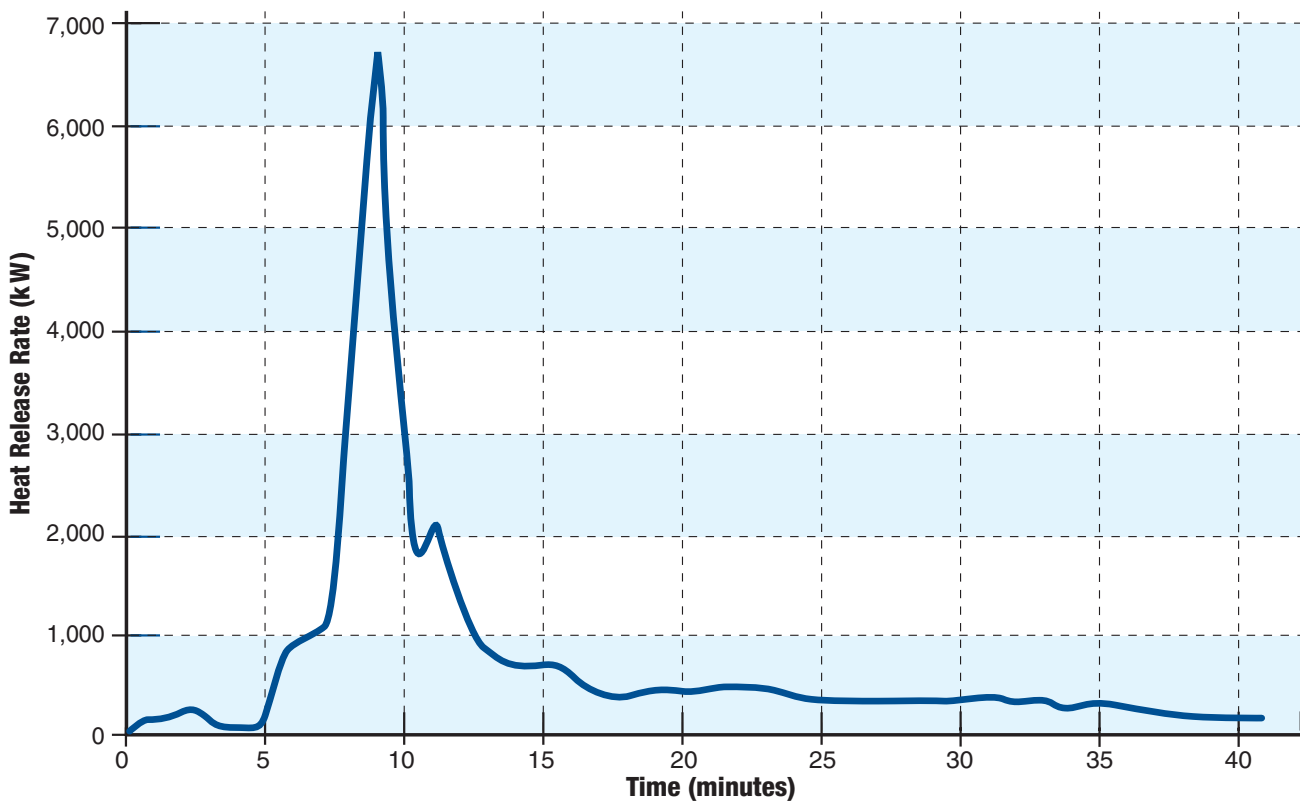


Figure A-1 Heat release rate for office module (Madrzykowski 1996).

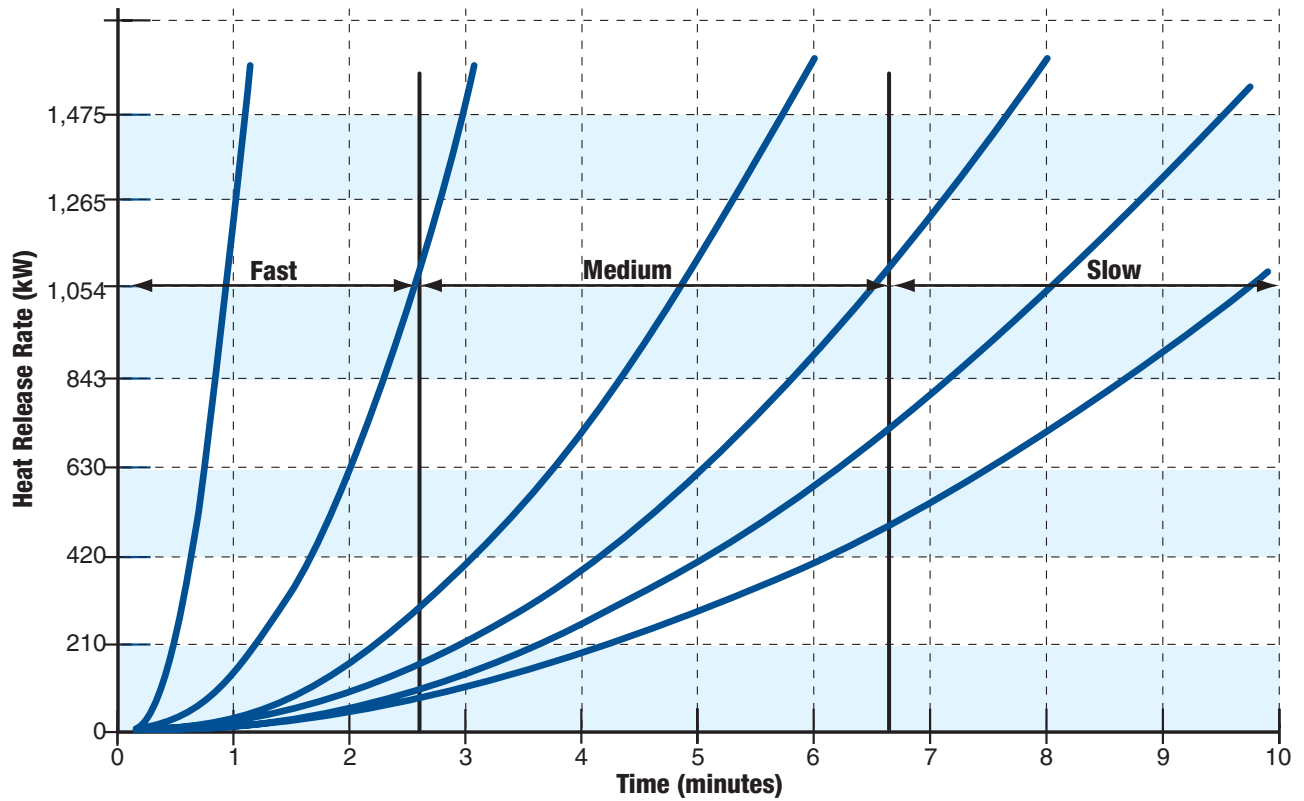


Figure A-2 Fire growth rates (from SFPE Handbook of Fire Protection Engineering).

Table A.1 Peak Heat Release Rates of Various Materials (NFPA 92B and NFPA 72)

Item	Heat Release Rate
Crumpled brown lunch bag, 6 g	1.2 kW
Folded double-sheet newspaper, 22 g	4 kW
Crumpled double sheet newspaper, 22 g	17 kW
Medium wastebasket with milk cartons	100 kW
Plastic trash bag with cellulosic material (1.2–14 kg)	120–350 kW
Upholstered chair with polyurethane foam	350 kW
Christmas tree, dry	500-650 kW
Latex foam mattress (heat at room door)	1,200 kW
Furnished living room	4,000–8,000 kW

After a fire has reached its peak heat release rate, it will decline after some period of time. At this point, most of the available fuel has typically been burned and the fire will slowly decrease in size. The length of the decay phase depends on what type of fuel is available, how complete was the combustion of the fuel, how much oxygen is present in the compartment, and whether any type of suppression is occurring. The burning rate of liquid fuels is on the order of  $50 \text{ g/m}^2\text{-s}$ , with an associated energy release rate per unit surface area of approximately  $2,000 \text{ kW/m}^2$ . The burning rate per unit area of information is useful to estimate the duration of a fire involving a particular fuel spread over a specified area.

### A.2.2 Stages of Fire Development

Generally, fires are initiated within a single fuel object. The smoke produced from the burning object is transported by a smoke plume and collects in the upper portion of the space as a layer. The smoke plume also transports the heat produced by the fire into the smoke layer, causing the smoke layer to increase in depth and also temperature. This smoke layer radiates energy back to unburned fuels in the space, causing them to increase in temperature.

Fire spreads to other objects either by radiation from the flames attached to the originally burning item or from the smoke layer. As other objects ignite, the temperature of the smoke layer increases further, radiating more heat to other objects. In small compartments, the unburned objects may ignite nearly simultaneously. This situation is referred to as “flashover.” In large compartments, it is more likely that objects will ignite sequentially. The sequence of the ignitions depends on the fuel arrangement, and composition and ventilation available to support combustion of available fuels.

### A.2.3 Behavior of Fully Developed Fires

A fully-developed fire is one that reaches a steady state burning stage, where the mass loss rate is relatively constant during that period. The equilibrium situation may occur as a result of a limited ventilation supply (in ventilation controlled fires) or due to characteristics of the fuel (fuel-controlled fires).

If the rate of mass burning based on the incident heat flux and fuel characteristics (see Section A.2.1) exceeds the amount that can be supported by the available air supply, the burning becomes ventilation controlled. Otherwise, the fire is referred to as being fuel controlled. The ventilation air for the fires may be supplied from openings to the room, such as open windows or doors, or other sources such as HVAC systems.

Given that the heat released per unit of oxygen is a relatively constant value of  $13.1 \text{ kJ/g}$  for common fuels, the air supply required to support fires of a particular heat release rate can be determined. For every  $1 \text{ MW}$  of heat release rate,  $76 \text{ g/s}$  of oxygen is consumed. Considering that air is 21 percent oxygen, this flow of oxygen requires a flow of  $0.24 \text{ m}^3/\text{s}$  ( $500 \text{ cfm}$ ) of ambient air. In the case of WTC 1 and WTC 2, for a  $3\text{-GW}$  fire, a flow of  $1,500,000 \text{ cfm}$  of air was required to support that fire. That airflow would have been supplied via openings in the exterior wall and the shaft walls.

Most of the research on fully-developed fires has been conducted in relatively small spaces with near-square floor plans. In such cases, the conditions (temperature of the smoke and incident heat on the enclosure) are relatively uniform throughout the upper portion of the space. However, Thomas and Bennetts (1999) have documented differences in that behavior for ventilation controlled fires in long, thin spaces or in large areas. In such cases, the burning occurs in the fuel nearest to the supply source of air. Temperatures are observed to be greatest nearest to the supply source of air.

In large or complex buildings, the incident flux on the structural elements is expected to vary over the entire space of fire involvement. A range of developing numerical models have the ability to compute the

variation of the fire imposed heat flux on a 3-dimensional grid. The Fire Dynamics Simulator from the National Institute of Standards and Technology (NIST) is an example of such a model that has the promise of developing into a tool that could be used to estimate the variation in incident heat flux on structural elements over a large space of fire involvement.

## A.3 Structural Response to Fire

### A.3.1 Effect of Fire on Steel

#### A.3.1.1 Introduction

Fire resistance is defined as the property of a building assembly to withstand fire, or give protection from it (ASTM 2001a). Included in the definition of fire resistance are two issues. The first issue is the ability of a building assembly to maintain its structural integrity and stability despite exposure to fire. Secondly, for some assemblies such as walls and floor-ceiling assemblies, fire resistance also involves serving as a barrier to fire spread.

Fire resistance is commonly assessed by subjecting a prototype assembly to a standard test. Results from the test are reported in terms of a fire resistance rating, in units of hours, based on the time duration of the test that the building assembly continues to satisfy the acceptance criteria in the test.

Fire resistance rating requirements for different building components are specified in building codes. These ratings depend on the type of occupancy, number of stories, and floor area. Because the standard test is intended to be a comparative test and is not intended to predict actual performance, the hourly fire resistance ratings acquired in the tests should not be misconstrued to indicate a specific duration that a building assembly will withstand collapse in an actual fire.

Generally, the fire resistance rating of a structural member is a function of:

- applied structural load intensity,
- member type (e.g., column, beam, wall),
- member dimensions and boundary end conditions,
- incident heat flux from the fire on the member or assembly,
- type of construction material (e.g., concrete, steel, wood), and
- effect of temperature rise within the structural member on the relevant properties of the member.

The fire performance of a structural member depends on the thermal and mechanical properties of the materials of which the building component is composed. As a result of the increase in temperature caused by the fire exposure, the strength of steel decreases along with its ability to resist deformation, represented by the modulus of elasticity. In addition, deformations and other property changes occur in the materials under prolonged exposures. Likewise, concrete is affected by exposure to fire and loses strength and stiffness with increasing temperature. In addition, concrete may spall, resulting in a loss of concrete material in the assembly. Spalling is most likely in rapid-growth fires, such as may have occurred in WTC 1 and WTC 2.

The performance of fire-exposed structural members can be predicted by structural mechanics analysis methods, comparable to those applied in ambient temperature design, except that the induced deformations and property changes need to be taken into consideration.

Beams and trusses may react differently to severe fire exposures, depending on the end conditions and fabrication. Unconnected members may collapse when the stresses from applied loads exceed the available

strength for beams and trusses. In the case of connected members, significant deflections may occur as a result of reduced elastic modulus, but structural integrity is preserved as a result of catenary action.

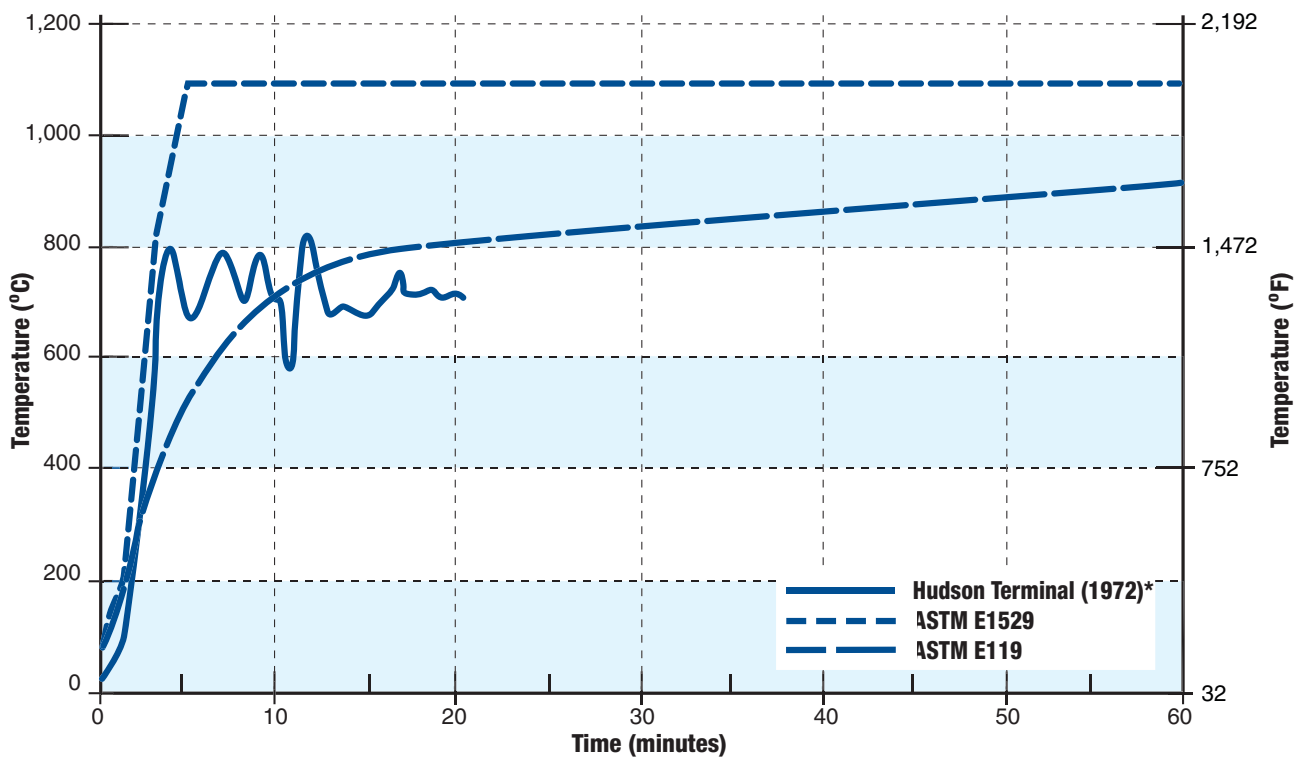
In the case of slender columns, the susceptibility for buckling increases with a decrease in the modulus of elasticity. Where connections of floor framing to columns fail, either at the ends or intermediate locations, column slenderness is increased, thereby increasing the susceptibility of a column to buckling.

Steels most often used in building design and construction are either hot-rolled or cold-drawn. Their strength depends mainly on their carbon content, though some structural steels derive a portion of their strength from a process of heat treatment known as quenching and tempering (e.g., ASTM A913 for rolled shapes and ASTM A325 and A490 for bolts).

### A.3.1.2 Evaluating Fire Resistance

#### Performance Criteria

Building code requirements for structural fire protection are based on laboratory tests conducted in accordance with ASTM E119, Standard Test Methods for Fire Tests of Building Construction and Materials (2000). In these tests, building assemblies, such as floor-ceilings, columns, and walls are exposed to heating conditions created in a furnace, following a specified time-temperature curve. In Figure A-3, time-



\*Hudson Terminal experiment conducted with normal office fuel load (6 psf) (DeCicco, et al. 1972)

Figure A-3 Comparison of exposure temperatures in standard tests.

temperature curves are presented for the standard fire exposure specified in ASTM E119, the standard hydrocarbon exposure in ASTM E1529, and a real building fire. As can be seen, each is somewhat different.

There are three performance criteria in the standard ASTM E119 test method. These are related to loadbearing capacity, insulation, and integrity:

1. **Loadbearing capacity:** For loadbearing assemblies, the test specimen shall not collapse in such a way that it no longer performs the loadbearing function for which it was constructed.
2. **Insulation:** For assemblies such as floors-ceilings and walls that have the function of separating two parts of a building,
  - a. the average temperature rise at the unexposed face of the specimen shall not exceed 139 °C (282 °F), and
  - b. the maximum temperature rise at the unexposed face of the specimen shall not exceed 181 °C (358 °F).
3. **Integrity:** For assemblies such as walls, floors, and roofs, the formation of openings through which flames or hot gases can pass shall not occur. Loss of integrity is deemed to have occurred when a specified cotton wool pad applied to the unexposed face is ignited.

Tests are conducted on prototype designs. The fire-resistance rating applies to replicates of the tested assembly, with limited changes permitted. Rules, guidelines, and correlations are available to assess the impact of changes or to develop acceptable variations to the design (ASCE/SFPE 1999).

### ASTM E119

The ASTM E119 test is a comparative test and is not intended to be predictive. The test fire exposure, while recognized as severe, is not representative of all fires. Heat transfer conditions associated with the exposing fire are different than those in actual fires. Further, the test is not a full-scale test, with no attempt to scale the response of the test specimen to actual size building assemblies. Although the test requires that floor-ceiling specimens be representative of actual building construction, achieving this in a 14-foot by 17-foot test specimen is difficult. Consequently, ASTM E119 is principally a thermal test, not a structural test, even though the test floor is loaded. Loading of floors and roofs is done to see if the fireproofing material will be dislodged by deflection and buckling of the steel during a fire.

Further, several factors are not applied in this test method, including structural framing continuity, member interaction, restraint conditions, and applied load intensity. The test only evaluates the performance of a building assembly, such as a wall or floor-ceiling assembly. The test does not consider the interaction between adjacent assemblies or the behavior of the structural frame. In “real” buildings, beam/girder/column connections range from simple shear to full moment connections and framing member size and geometry vary significantly, depending on the structural system and building size and layout.

In the Underwriters Laboratories, Inc. (UL) version of the ASTM E119 test, UL 263, the beams are placed on shelf angles and steel wedges are driven by sledgehammers between the end of the beam and the heavy massive steel and concrete furnace frame. This is referred to as a “restrained beam,” and the fire test results are published in Volume 1 of the UL Fire Resistance Directory, which is the major reference used by architects and engineers to select designs that meet the building code requirements for fire resistance ratings. The UL Fire Resistance Directory also publishes unrestrained fire resistance ratings based on critical temperature rise in the steel member as discussed in Section A.3.1.6. In spite of the ASTM E119 test limitations relative to the structural conditions that exist in real buildings, the fire test is conservative to the

point that more fire protection material is required than has been demonstrated necessary in large scale fire tests conducted and reported in the international fire research literature.

There has been much interest in revising the ASTM E119 Standard Fire Test. Arguments are posed that the fire exposure is too severe, while others suggest that the fire exposure is not severe enough. A good compromise is a performance oriented analysis using design fire curves for very specific occupancies and building geometry while still permitting the use of ASTM E119 for general applications.

For most of the 1900s, there was a single U.S. standard time-temperature curve described by ASTM E119. Most of the world adopted that curve or one similar in running the test furnaces.

In 1928, Ingberg of the National Bureau of Standards published a paper on the severity of fire (Ingberg 1928) in which he equated the gross combustible fuel load (combustible content in mass per unit area) to the potential fire exposure in terms of duration of exposure to a fire following the standard (ASTM E119) fire curve. Although subsequent research has shown the simple relationship proposed by Ingberg holds only in limited cases where the fire ventilation is the same as that present in his test series, his equation is still widely published in texts and used as the basis of regulation.

In the 1950s and 1960s, it was demonstrated that, for severe, fully involved fires, the intensity and duration of burning within compartments and other enclosures were also functions of the availability of air for combustion, commonly referred to as ventilation and normally coming from openings such as doors and broken windows or from forced ventilation from the HVAC system.

In Sweden, an extensive family of fire curves has been developed, by test, for fully involved (i.e., post flashover) fires as a combined function of fuel load and ventilation (Magnusson and Thelandersson 1970). The published curves have peak temperatures of 600–1,100 °C (1,100–2,000 °F).

Most recently, Ian Thomas in Australia has demonstrated with reduced scale models that the combustion process in facilities where there is a depth from the vent opening (e.g., broken windows) to the actual fuel can produce conditions where a large portion of the vaporized fuel actually burns at a point removed from the location of the solid fuel (combustible material) source. Thomas' experiments used fully involved spaces where the depth from the vent opening was at least twice the width of the test space. In these experiments, the air supply drawn into the test space by the fire was insufficient to burn all of the available fuel. Fuel once vaporized was transported to the openings and burned there, producing an unexpectedly high heat flux on the elements at and near the vent opening. The importance of Thomas' work is that it demonstrates the fact that, in many fires, the reality is that the fire exposing the structural elements is not necessarily a constant in either time or space.

Fortunately, there are now advanced numerical models capable of describing the fire caused environment in detail.

#### **ASTM E1529 and UL 1709: The Hydrocarbon Pool Curves**

In the late 1980s, as a result of failures of fireproofed steel members exposed to petroleum spill fires, the petroleum industry felt a need to develop a new test curve. The curve developed was designed to apply a sudden and intense shock, typified by a large hydrocarbon pool fire either burning in the open or in some other situation where there was no significant restraint to the flow of combustion air to the burning pool fire. ASTM E1529 was developed to answer this need. The objective of this ASTM test is to almost instantaneously impose 158 kW/m<sup>2</sup> (50,000 Btu/ft<sup>2</sup>-hr) on the element under test. Additionally a similar but somewhat more severe test procedure has been developed by Underwriters Laboratories and published as their standard UL 1709. The UL test is designed to impose 200 kW/m<sup>2</sup> (65,000 Btu/ft<sup>2</sup>-hr) on the test element. This unusual difference in the ASTM and UL standards reflects a technical difference of opinion



between the two organizations. The tests are often quoted as a time-temperature curve quickly reaching and maintaining a test furnace temperature of 1,093 °C (2,000 °F) in the case of the ASTM standard and 1,143 °C (2,089 °F) at UL. The hydrocarbon time-temperature curve is, however, actually a test-specific item and can vary some from test apparatus to test apparatus.

The ASTM E119 curve was derived from experiments and is empirically based; however, ASTM E1529 exposure is based on judgment, experience, and a database of experiments concerning the measurement of the temperatures involved in large hydrocarbon fires. The incident flux approximates the incident flux on a member completely bathed in the flame from a large free-burning pool fire. Although both of the ASTM curves are useful in conducting tests of fireproofed building elements as pre-installment tests, they are not predictions of the intensity of actual fires and are often not appropriate as an input to models or other computations seeking to assess a fire hazard for a building.

A prime impact of the high flux “shock” exposure is to test the capability of the fireproofing to survive such exposure. In addition, such thermal shock could induce spalling in concrete systems.

Comparison between ASTM E119, ASTM E1529, and UL 1709 is further complicated by instrumentation differences in the two “hydrocarbon fire” tests and that used in the ASTM E119 test. In particular, different thermocouple installations are used to control and record furnace temperatures in the respective tests. In the ASTM E119 test, the thermocouples are contained within a protective capped steel pipe, resulting in a time delay between the actual and recorded furnace temperatures. In the hydrocarbon tests, the thermocouples are bare, thereby providing a more timely indication of the actual gas temperature. The lag in ASTM E119 is most pronounced at the start of the test. Figure A-3 provides a plot of the two standard curves with an additional curve of the approximate actual temperature (if measured with bare thermocouples) in an ASTM E119 furnace test. Most of the tests to date have been conducted using the UL 1709 curve. Many tested items show a significantly shorter time to failure using the UL 1709 procedure as compared to the ASTM E119 procedure.

### **A.3.1.3 Response of High-rise, Steel-frame Buildings in Previous Fire Incidents**

In recent years, three notable fires have occurred in steel frame buildings, though none involved the total floor area as in WTC 1 and WTC 2. However, prior to September 11, 2001, no protected steel frame buildings had been known to collapse due to fire. These previous three fire incidents include the following:

- 1<sup>st</sup> Interstate Bank Building, Los Angeles, May 4-5, 1988
- Broadgate Phase 8, UK, 1990
- One Meridian Plaza, Philadelphia, February 23-24, 1991

The steel in the 1st Interstate Bank Building and One Meridian Plaza was protected with spray-applied protection. Because the fire occurred at the Broadgate complex while it was under construction, the steel beams had not yet been protected. The fire durations of the three incidents are indicated in Table A.2. The durations noted in the table refer to the overall duration of the incident. The fire duration in a particular area of the building was likely less than that noted.

In the case of the fire at One Meridian Plaza, the fire burned uncontrolled for the first 11 hours and lasted 19 hours. Contents from nine floors were completely consumed in the fire. In addition to these experiences in fire incidents, as a result of the Broadgate fire, British Steel and the Building Research Establishment performed a series of six experiments at Cordington in the mid-1990s to investigate the behavior of steel frame buildings. These experiments were conducted in a simulated, eight-story building. Secondary steel beams were not protected. Despite the temperature of the steel beam reaching 800–900 °C

Table A.2 Fire Duration in Previous Fire Incidents in Steel-frame Buildings

Building	Date	Fire Duration (hours)
1st Interstate Bank Building	May 4-5, 1988	3.5
Broadgate Phase 8	1990	4.5
One Meridian Plaza	February 23-24, 1991	19 (11 uncontrolled)

(1,500–1,700 °F) in three tests (well above the traditionally assumed critical temperature of 600 °C [1,100 °F]), no collapse was observed in any of the six experiments.

One important aspect of these previous incidents is that the columns remained intact and sustained their load carrying ability throughout the fire incidents (though there was no structural damage caused by impacts). Throughout the fire in One Meridian Plaza, horizontal forces were exerted on the columns by the girders and despite the 24- to 36-inch deflections of the girders, floor beams, and concrete and steel deck floor slabs, the columns continued to stabilize the building throughout the fire and for several years after the fire.

Questions have been raised about the comparison of the structural performance of the WTC 1 and WTC 2 and the Empire State Building. In the case of the Empire State Building:

1. The impacting aircraft was a U. S. Army Air Force B-25 bomber weighing 12 tons with a fuel capacity of 975 gallons, which, at the time of the crash, was traveling at a speed estimated to be 250 mph;
2. Crash damage to structural steel was confined to three steel beams. One exterior wall column withstood the direct impact without visible effect;
3. Exterior walls are ornamental cast aluminum panels under windows with steel trim backed by 8 inches of brick. The walls at columns are 8 inches of limestone backed by 8 inches of brick supported on steel framing; and
4. The floors above the Saturday morning plane crash were largely vacant and unoccupied, so the fire load was minimal and perhaps close to zero. Fire was confined to a portion of two floors. Because the building had few occupants at the time of the crash, the fire department could concentrate on controlling and extinguishing the fire.

#### A.3.1.4 Properties of Steel

The principal thermal properties that influence the temperature rise and distribution in a member are its thermal conductivity, specific heat, and density. The temperature-dependence of the thermal conductivity and specific heat for steel are depicted in Figure A-4.

The mechanical properties that affect the fire performance of structural members are strength, modulus of elasticity, coefficient of thermal expansion, and creep of the component materials at elevated temperatures. Information on the thermal and mechanical properties at elevated temperatures for various types of steel is available in the literature (Lie 1992, Milke 1995, Kodur and Harmathy 2002).

References to the tensile or compressive strength of steel relate either to the yield strength or ultimate strength. Figure A-5 shows the stress-strain curves for a structural steel (ASTM A36) at room temperature and elevated temperatures. As indicated in the figure, the yield and ultimate strength decrease with temperature as does the modulus of elasticity. Figure A-6 shows the variation of strength with temperature

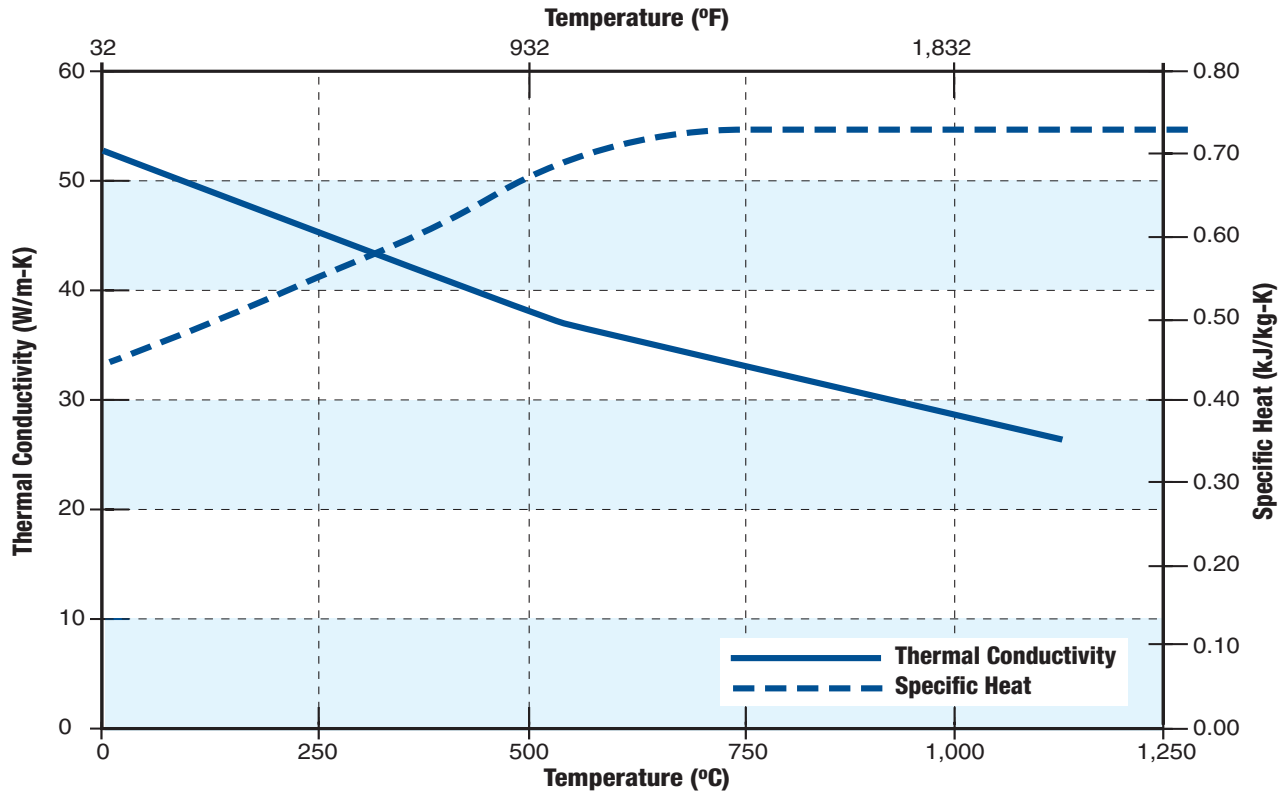


Figure A-4 Thermal properties of steel at elevated temperatures (SFPE 2000).

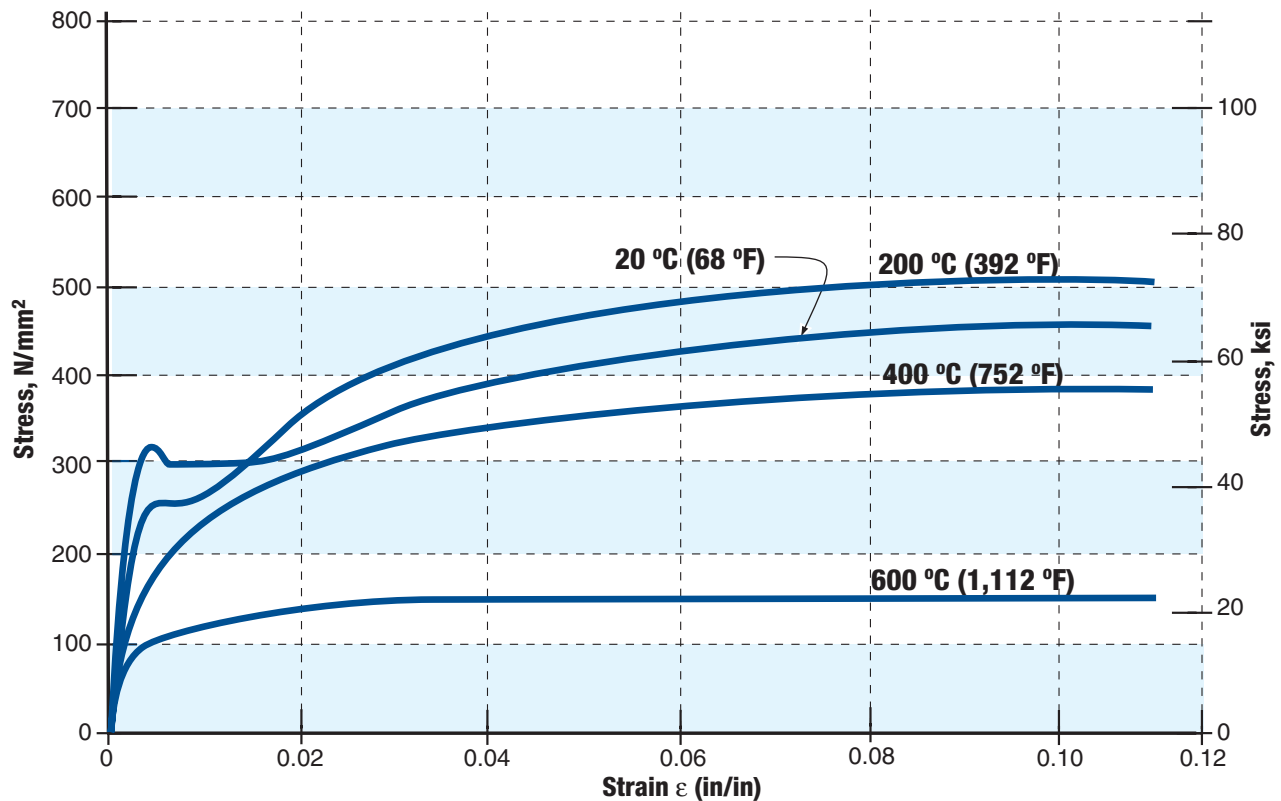


Figure A-5 Stress-strain curves for structural steel (ASTM A36) at a range of temperatures (SFPE 2000).

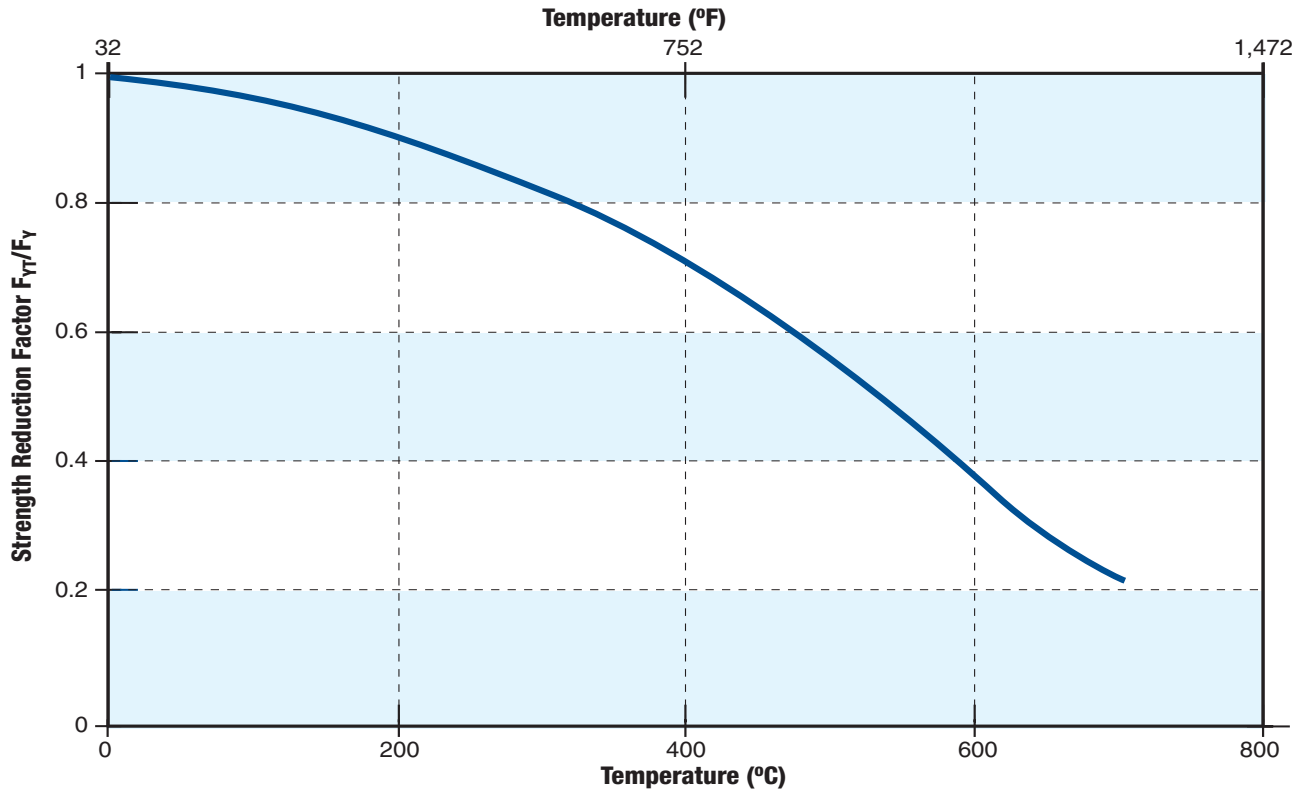


Figure A-6 Strength of steel at elevated temperatures (Lie 1992).

(ratio of strength at elevated temperature to that at room temperature) for hot rolled steel such as A36. As indicated in the figure, if the steel attains a temperature of 550 °C (1,022 °F), the remaining strength is approximately half of the value at ambient temperature.

The modulus of elasticity,  $E_0$ , is about  $210 \times 10^3$  MPa for a variety of common steels at room temperature. The variation of the modulus of elasticity with temperature for structural steels and steel reinforcing bars is presented in Figure A-7. As in the case of strength, if the steel attains a temperature of 550 °C (1,022 °F), the modulus of elasticity is reduced to approximately half of the value at ambient temperature.

Figure A-8 shows the variation of yield strength of light gauge steel at elevated temperatures, corresponding to 0.5 percent, 1.5 percent, and 2 percent strains based on the relationships in Gerlich (1995), Makelainen and Miller (1983), and BSI (2000).

In addition to the changes in the properties with increasing temperature, steel expands with increasing temperature. The coefficient of thermal expansion for structural steel is approximately  $11 \times 10^{-6}$  mm/mm-°C. Consequently, an unrestrained, 20-meter-long steel member that experiences a temperature increase of 500 °C (1,022 °F) will expand approximately 110 mm. WTC 5 had many buckled girders and beams on the burned-out fire floors where the expansion was restrained.

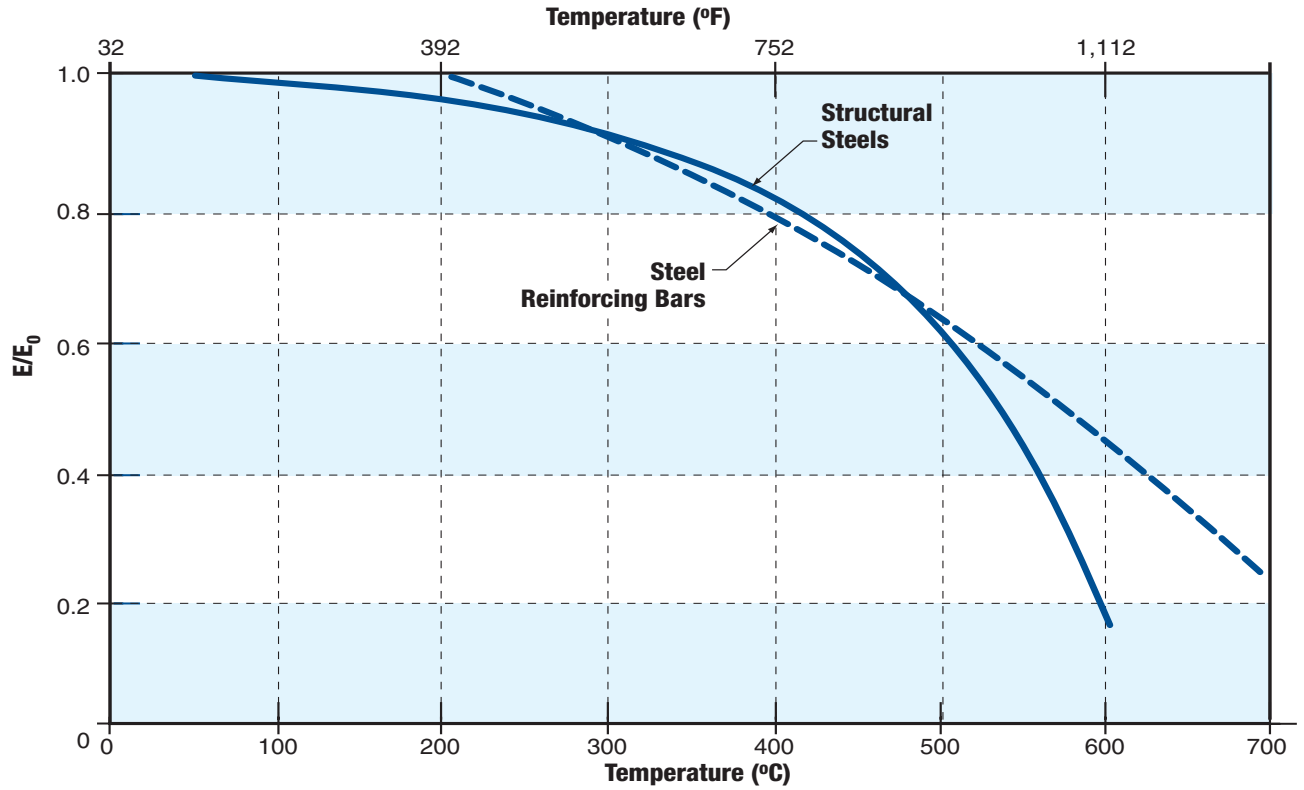


Figure A-7 Modulus of elasticity at elevated temperatures for structural steels and steel reinforcing bars (SFPE 2000).

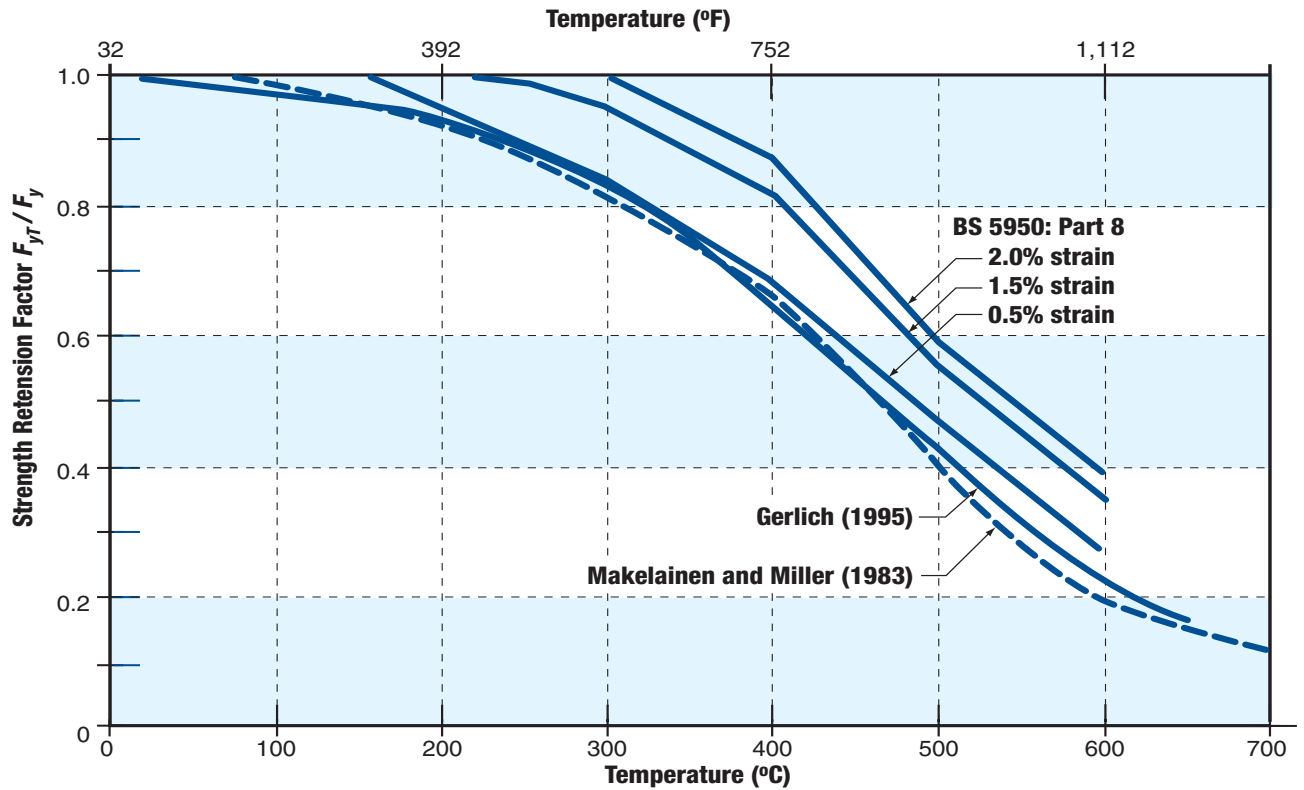


Figure A-8 Reduction of the yield strength of cold-formed light-gauge steel at elevated temperatures.

An approximate melting point for steel is 1,400 °C (2,500 °F); however, the melting temperature for a particular steel component varies with the steel alloy used.

#### A.3.1.5 Fire Protection Techniques for Steel

Given the significant reduction in the mechanical properties of steel at temperatures on the order of 540 °C (1,000 °F), isolated and unprotected steel members subjected to the standard test heating environment are only able to maintain their structural integrity for 10 to 20 minutes, depending on the mass and size of the structural member. Unprotected open web steel joists supporting concrete floors in the ASTM E119 fire test have been tested and collapse in 7 minutes (Wang and Kodur 2000).

Isolated and unprotected steel box columns 8 inches x 6 1/2 inches formed using 1/4-inch plate and channels in an ASTM E119 fire test collapse in about 14 minutes (Kodur and Lie 1995). Consequently, measures are taken to protect loadbearing, steel structural members where the members are part of fire resistant assemblies. A variety of methods are available to limit the temperature rise of steel structural members, including the insulation method and the capacitive method.

**Insulation Method:** The insulation method consists of attaching insulating spray-applied materials, board materials, or blankets to the external surface of the steel member. A variety of insulating materials have been used following this method of protection, including mineral-fiber or cementitious spray-applied materials, gypsum wallboard, asbestos, intumescent coatings, Portland cement concrete, Portland cement plaster, ceramic tiles, and masonry materials. The insulation may be sprayed directly onto the member being protected, such as is commonly done for steel columns, beams, or open web steel joists. The spray-applied mineral fiber, fire resistive coating is a factory mixed product consisting of manufactured inorganic fibers, proprietary cement-type binders, and other additives in low concentrations to promote wetting, set, and dust control. Air setting, hydraulic setting, and ceramic setting binders can be used in varying quantities and combinations or singly, depending on the particular application.

Alternatively, the insulation may be used to form a “membrane” around the structural member, in which case a fire resistive barrier is placed between a potential fire source and the steel member. An example of membrane protection is a suspended ceiling positioned below open web steel joists. (In order for a suspended ceiling assembly to perform effectively as a membrane form of protection, it must remain in place despite the fire exposure. Only some suspended ceiling assemblies have this capability.)

In most of the WTC complex buildings and tall buildings built over the last 50 years, the preferred method has been spray-applied mineral fiber or cementitious materials. Of these 50 years, for the first 20 years the product contained asbestos and for the last 30 years it has been asbestos free. The WTC 1, 2, and 7 incidents are the first known collapses of fire resisting steel frame buildings protected with this type of fireproofing material. Occasionally, a portion of the steel is protected with a spray or trowel applied plaster or Portland cement (e.g., Gunitite or shotcrete).

**Capacitive Method:** The capacitive heat sink method is based on the principle of using the heat capacity of a protective material to absorb heat. In this case, the supplementing material absorbs the heat as it enters the steel and acts as a heat sink. Common examples include concrete filled hollow steel columns and water filled hollow steel columns (Kodur and Lie 1995). In addition, a concrete floor slab may act as a heat sink to reduce the temperature of a supporting beam or open web steel joist.

#### A.3.1.6 Temperature Rise in Steel

In building materials such as steel, a critical temperature is often referenced at which the integrity of fully-loaded structural members becomes questionable. The critical temperature for steel members varies with the type of steel structural member (e.g., beams, columns, bar joists, or reinforcing steel). North

American Test Standards (e.g., ASTM E119) assume a critical temperature of 538 °C (1,000 °F) for structural steel columns. The critical temperatures for columns and other steel structural elements are given in Table A.3. The critical temperature is defined as approximately the temperature where the steel has lost approximately 50 percent of its yield strength from that at room temperature. In an actual structure, the actual impact of such heating of the steel will also depend on the actual imposed load, member end restraint (axial and rotational), and other factors as discussed in Section A.3.1.7.

**Table A.3 Critical Temperatures for Various Types of Steel**

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

To limit the loss of strength and stiffness, external fire protection is provided to the steel structural members to satisfy required fire resistance ratings. This is usually achieved by fire protecting the steel members to keep the temperature of the steel, in case of a fire, from reaching a critical limit. Traditionally, the amount of fire protection needed is based on the results of standard fire resistance tests.

The temperature attained in a fire-exposed steel member depends on the fire exposure, characteristics of the protection provided, and the size and mass of the steel. For steel members protected with direct-applied insulating materials, the role of the insulating materials is strongly dependent on their thermal conductivity and thickness.

The role of the fire exposure and size and mass of the steel can be demonstrated by analyzing the temperature rise in two protected steel columns with two different fire exposures. For this comparative analysis, the fire exposure associated with two standard fire resistance tests is selected, ASTM E119 and UL 1709. The following two column sizes are selected for this comparative analysis:

- W14X193
- steel box column, 36 inches x 16 inches, with a wall thickness of 7/8 inch for the 36-inch-wide side and 15/16 inch for the 16-inch-wide side

In the first analysis, the steel columns are considered to be unprotected. The results of the analysis are presented in Figure A-9. In the second analysis, 1 inch of a spray-applied, mineral fiber insulation material was assumed to be present (the thermal conductivity of the insulation material was assumed to be 0.116 W/m-K). The results of this analysis are presented in Figure A-10.

In both analyses, the resulting steel column temperatures follow expected trends. The more massive column (the tube) experiences less temperature rise for the same fire exposure than the lighter column (the W14x193). The unprotected columns reach critical temperatures exposed to ASTM E119 condition in 15 to 18 minutes. For the more severe UL 1709 exposure, the unprotected columns reach critical temperatures in 6 to 7 minutes. In contrast, the temperature of the protected columns after 2 hours of exposure to the ASTM E119 conditions is 240 °C (464 °F) for the tube, while the temperature of the W14x193 is 330 °C (626 °F). For the more severe fire exposure associated with UL 1709, the temperature of the steel columns after 2 hours is 60–80 °C (140–176 °F) greater than for each of the steel columns exposed to the ASTM E119 conditions.

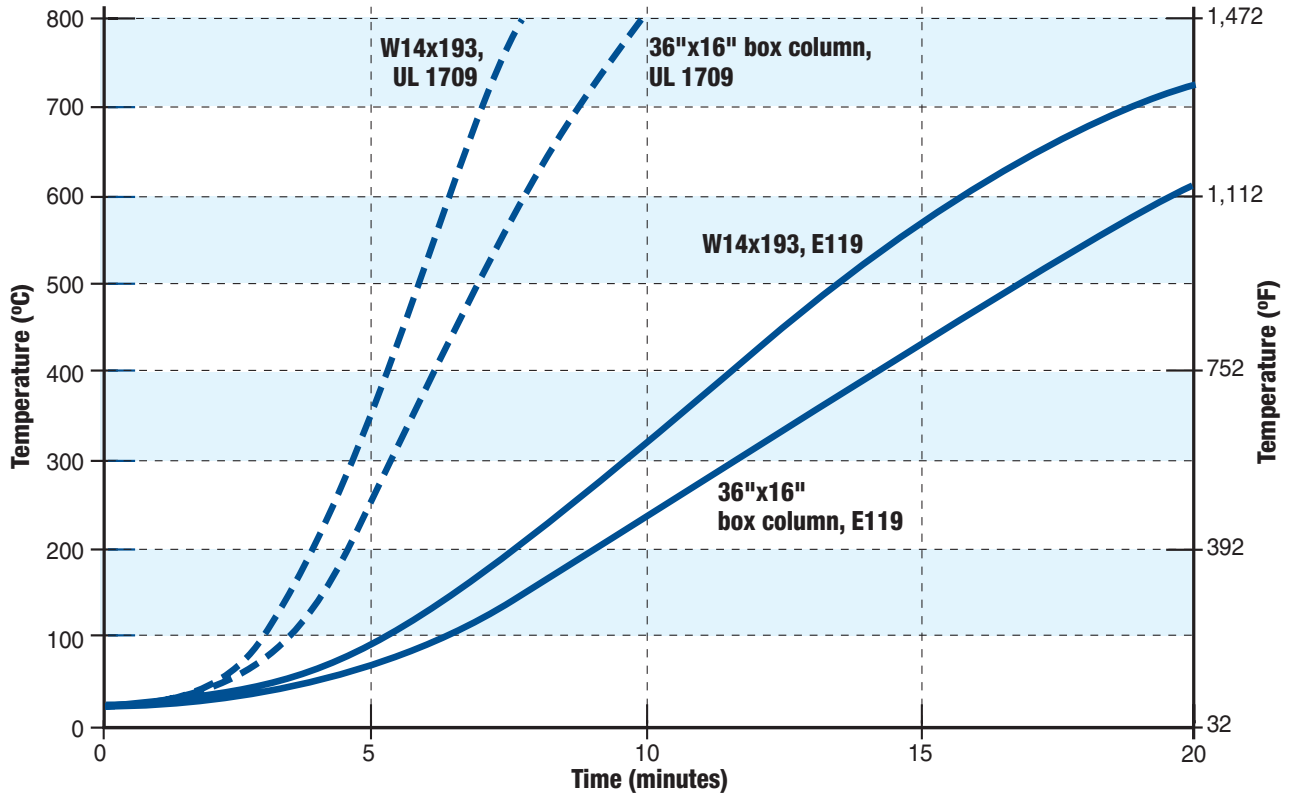


Figure A-9 Steel temperature rise due to fire exposure for unprotected steel column.

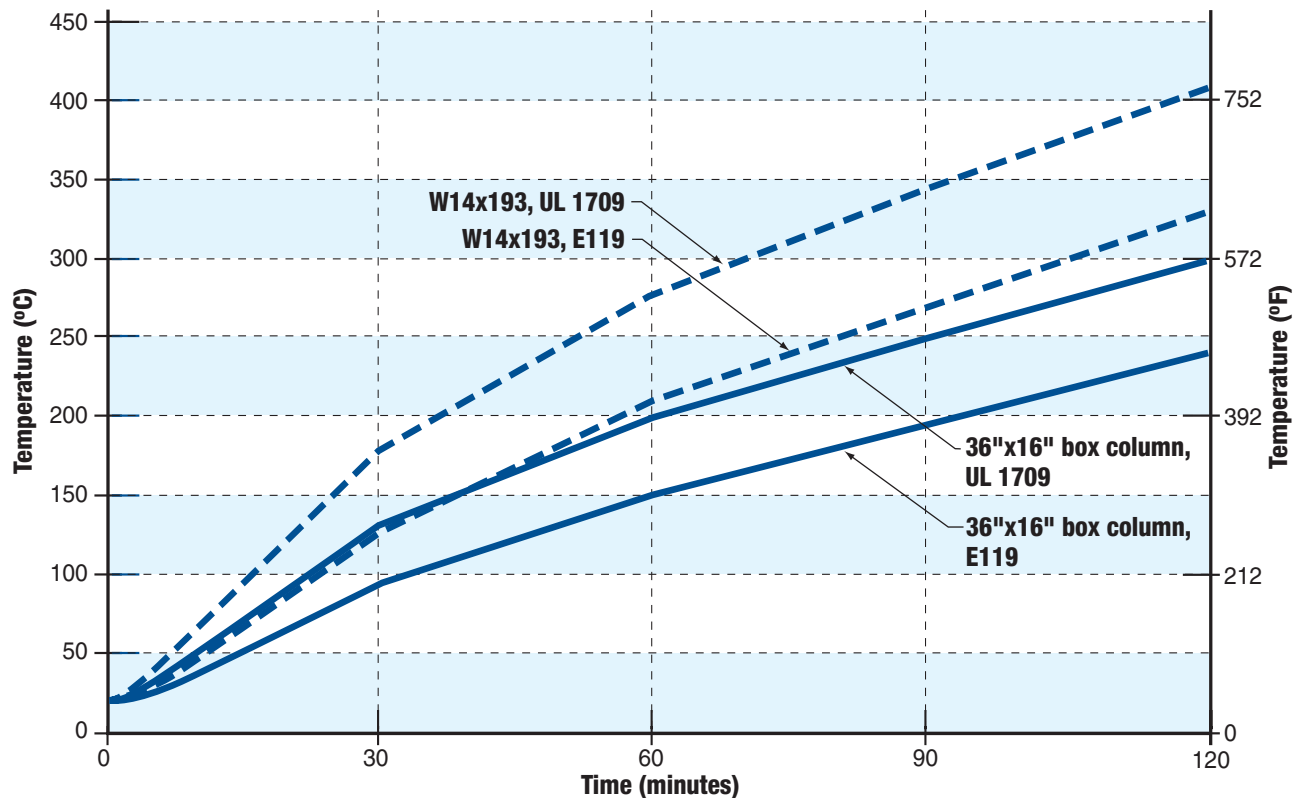


Figure A-10 Steel temperature rise due to fire exposure for steel column protected with 1 inch of spray-applied fireproofing.



Fully developed building fires can generally attain average gas temperatures throughout the room containing the fire in excess of 1,000 °C (1,800 °F). The temperature measurements acquired in experiments involving office furnishings conducted by DeCicco, et al. (1972) in the Hudson Terminal Building (30 Church Street, New York), along with the two time-temperature curves from the standard tests is presented in Figure A-3. Temperature development in the first 5 minutes in the room space is notably similar in the experiment with that in ASTM E1529, UL 1709, and the bare thermocouple temperatures for ASTM E119.

Greater temperatures may be acquired locally in a room and especially within flames. Research has indicated that, in the center of flames generated by relatively small fires, temperatures may approach 1,300 °C (2,400 °F) (Baum and McCaffrey 1988). For larger fires, where radiation losses may be reduced, it is conceivable that fire temperatures could reach 1,400 °C (2,550 °F), although this has not been confirmed experimentally.

### **A.3.1.7 Factors Affecting Performance of Steel Structures in Fire**

Several factors influence the behavior of steel structures exposed to fire. The more significant factors are discussed in the following sections.

**Loading:** One of the major factors that influence the behavior of a structural steel member exposed to fire is the applied load (Fitzgerald 1998, Lie 1992). A loss of structural integrity is expected when the applied loading exceeds or is equal to the ultimate strength of the member. The limiting temperature and the fire resistance of the member increases if the applied load decreases. Traditional fire resistance tests apply a load that results in the maximum allowable stress on the structural member resistance.

**Connections:** Beam-to-column connections in modern steel-framed buildings may be either of bolted or welded construction, or a combination of these types. Most are designed to transmit shears from the beam to the column, although some connections are designed to provide flexural restraint between the beam and column, as well, in which case they are termed “moment resisting.” When moment-resisting connections are not provided in a building, diagonal bracing or shear walls must be provided for lateral stability. When fire-induced sagging deformations occur in simple beam elements with shear connections, the end connections provide restraint against the induced rotations and develop end moments, reducing the mid-span moments in the beams, as well as the tensile catenary action. The moment and tension resisted by connections reduces the effective load ratio to which the beams are subjected, thereby enhancing the fire resistance of the beams as long as the integrity of the connection is preserved. This beneficial effect is more pronounced in large multi-bay steel frames with simple connections. Connections are generally not included as part of the assembly tested in traditional fire resistance tests. Further, most modeling efforts assume that the pre-fire characteristics of a connection are preserved during the fire exposure.

The investigating team observed damaged connections in WTC 5. For example, distorted bolts and bolt holes were found. The performance of connections seem to often determine whether a collapse is localized or leads to progressive collapse. In the standard fire tests of structural members, the member to be tested is wedged into a massive restraining frame. No connections are involved. The issue of connection performance under fire exposure is critical to understanding building performance and should be a subject of further research.

**End Restraint:** The structural response of a steel member under fire conditions can be significantly enhanced by end restraints (Gewain and Troup 2001). For the same loading and fire conditions, a beam with a rotational restraint at its ends deflects less and survives longer than its simply supported, free-to-expand counterpart. The addition of axial restraint to the end of the beam results in an initial increase in the deflections, due to the lack of axial expansion relief. With further heating, however, the rate of increase in deflection slows.

**Effectiveness of Fireproofing:** The acceptability of a particular fireproofing material as an insulator is examined as part of ASTM E119. The fireproofing material should form a stable thickness of insulating cover for the steel. Mechanical or impact damage to the fireproofing material prior to the fire exposure that results in a loss of insulating material reduces the ability of the material to act as an insulator (Ryder, et al. 2002). During the fire exposure in the ASTM E119 tests, fireproofing material may fall off as a result of thermal strains caused by differing amounts of expansion in the fireproofing and steel, excess curvature of the steel, or decomposition of the fireproofing material. If the fall-off occurs early in the test or fire exposure, the performance of the assembly is likely to be unsatisfactory. However, if the fireproofing material falls off late in the test or at the time when the fire is declining in intensity, the impact of the lost protection may not be significant. Several test methods other than ASTM E119 can be followed to assess the performance characteristics of fireproofing material. These tests are indicated in Table A.4.

Both the sprayed fiber and, to a lesser extent, cementitious materials, can sometimes fail to adhere to the steel, be mechanically damaged, or otherwise be degraded when exposed to a fire. The current quality control testing of adhesion/cohesion and density, while helpful, does not solve the problem of assuring that the fireproofing will be present at the time of a fire and function throughout the duration of the fire exposure. Other factors that can affect the durability and performance of fireproofing include resistance to abrasion, shock, vibration, and high temperatures.

**Sprinklers:** Sprinkler systems can be very effective in protecting all structures from the effects of fire. Automatic sprinkler systems are considered to be an effective and economical way to apply water promptly to control or suppress a fire. In the event of fire in a building, the temperature rise in the structural members located in the vicinity of sprinklers is limited. Therefore, the fire resistance of such members is enhanced. The sprinkler piping is sized considering all sprinklers in a design area of operation that are discharging water. For office buildings, typical areas of operation are approximately 1,500 to 2,500 square feet. Should a fire involve an area larger than the area of operation, the water supply may be overwhelmed, thereby negatively impacting the effectiveness of the sprinkler system.

**Table A.4 Test Methods for Spray-applied Fireproofing Materials**

<b>Standard</b>	<b>Title</b>
ASTM E605	<i>Thickness and Density of Sprayed Fire-Resistive Materials Applied to Structural Members.</i>
ASTM E736	<i>Cohesion/Adhesion of Sprayed Fire-Resistive Materials Applied to Structural Members</i>
ASTM E759	<i>Effect of Deflection of Sprayed Fire-Resistive Materials Applied to Structural Members</i>
ASTM E760	<i>Effect of Impact on the Bonding of Sprayed Fire-Resistive Materials Applied to Structural Members</i>
ASTM E761	<i>Compressive Strength of Sprayed Fire-Resistive Materials Applied to Structural Members</i>
ASTM E859	<i>Air Erosion of Sprayed Fire-Resistive Materials Applied to Structural Members</i>
ASTM E937	<i>Corrosion of Steel by Sprayed Fire-Resistive Materials Applied to Structural Members</i>

**Structural Interaction:** In contrast to an isolated member exposed to fire, the way in which a complete structural building frame performs during a fire is influenced by the interaction of the connected structural members in both the exposed and unexposed portions of the building. This is beneficial to the overall behavior of the complete frame, because the collapse of some of the structural members may not necessarily endanger the structural stability of the overall building. In such cases, the remaining interacting members develop an alternative load path to bridge over the area of collapse. This is a current area of research and is not addressed by traditional fire resistance tests.

**Tensile Membrane Action:** A tensile membrane (catenary) action can be developed by metal deck and reinforced concrete floor slabs in a steel-framed building whose members are designed and built to act compositely with the concrete slab (Nwosu and Kodur 1999). This action occurs when the applied load on the slab is taken by the steel reinforcement, due to cracking of the entire depth of concrete cross-section or heating of supporting steel members beyond the critical temperature. Tensile membrane action enhances the fire resistance of a complete framed building by providing an alternative load path for structural members that have lost their loadbearing capacity.

**Temperature Distribution:** Depending on the protective insulation and general arrangements of members in a structure, steel members will be subjected to temperature distributions that vary along the length or over the cross-section. Members subjected to temperature variation across their sections may perform better in fire than those with uniform temperature. This is due to the fact that sections with uniform temperatures will attain their load capacity at the same time. However, in members subjected to non-uniform temperature distribution, a thermally induced curvature will occur to add to the deflections due to applied loads and some parts will attain the load limit before the others. Temperature distributions within structural members may be attained if the member is part of a wall or floor-ceiling assembly where the fire exposure is applied only to one side.

### A.3.2 Effect of Fire on Concrete

#### A.3.2.1 General

Concrete is one of the principal materials widely used in construction and, in fire protection engineering terminology, is generally classified as Group L (loadbearing) building material: materials capable of carrying high stresses. The word concrete covers a large number of different materials, with the single common feature that they are formed by the hydration of cement. Because the hydrated cement paste amounts to only 24 to 43 volume percent of the materials present, the properties of concrete may vary widely with the aggregates used.

Traditionally, the compressive strength of concrete used to be around 20-50 MPa, which is referred to as normal-strength concrete. Depending on the density, concretes are usually subdivided into two major groups: (1) normal-weight concrete, made with normal-weight aggregate, with densities in the 2,200 to 2,400 kg/m<sup>3</sup> range, and (2) lightweight concrete, made with lightweight aggregate, with densities between 1,300 and 1,900 kg/m<sup>3</sup>.

The floor slabs at WTC 1 and WTC 2 (as well as in most of the WTC buildings and vicinity) were made of concrete made of metal deck. The floor construction typically consisted of 4 inches of lightweight concrete fill on corrugated metal deck. Hence, the discussion here is focused on lightweight concrete.

#### A.3.2.2 Properties of Lightweight Concrete

As with steel, concrete loses strength with temperature, though some concretes maintain their ambient temperature strength up to a greater temperature than structural steel. Some lightweight concretes may not

exhibit the same level of performance as normal weight concretes under severe fire conditions. In these concretes, spalling under fire conditions is one of the major concerns. The fire resistance of lightweight concrete structural members is dependent on spalling characteristics in addition to thermal and mechanical properties of lightweight concrete at elevated temperatures.

A great deal of information is available in the literature on the properties of lightweight concrete (Abrams 1979, ACI 1989, Lie 1992, Kodur and Harmathy 2002). The modulus of elasticity ( $E$ ) of various concretes at room temperature may fall within a very wide range,  $5.0 \times 10^3$  to  $50.0 \times 10^3$  MPa, dependent mainly on the water-cement ratio in the mixture, the age of concrete, and the amount and nature of the aggregates. The modulus of elasticity decreases rapidly with the rise of temperature, and the fractional decline does not depend significantly on the type of aggregate (Kodur 2000)(see Figure A-11;  $E_0$  in the figure is the modulus of elasticity at room temperature).

The compressive strength ( $\sigma_u$ ) of lightweight concrete can vary within a wide range and is influenced by the same factors as the modulus of elasticity. For conventionally produced lightweight concrete (at the time of the WTC construction in 1970s), the strength at room temperature usually was in the 20 to 40 MPa range. The variation of the compressive strength with temperature is presented in Figure A-12 for two lightweight aggregate concretes, one of which is made with the addition of natural sand (Kodur 2000); ( $\sigma_u$ )<sub>0</sub> in the figures refers to the compressive strengths of concrete at room temperature). The strength decrease is minimal up to about 300 °C (570 °F); above these temperatures, the strength loss is significant.

Generally, lightweight concrete has a lower thermal conductivity, lower specific heat, and lower thermal expansion at elevated temperatures than normal-strength concrete. As an illustration, the usual ranges of variation of the specific heat for normal-weight and lightweight concretes are shown in Figure A-13.

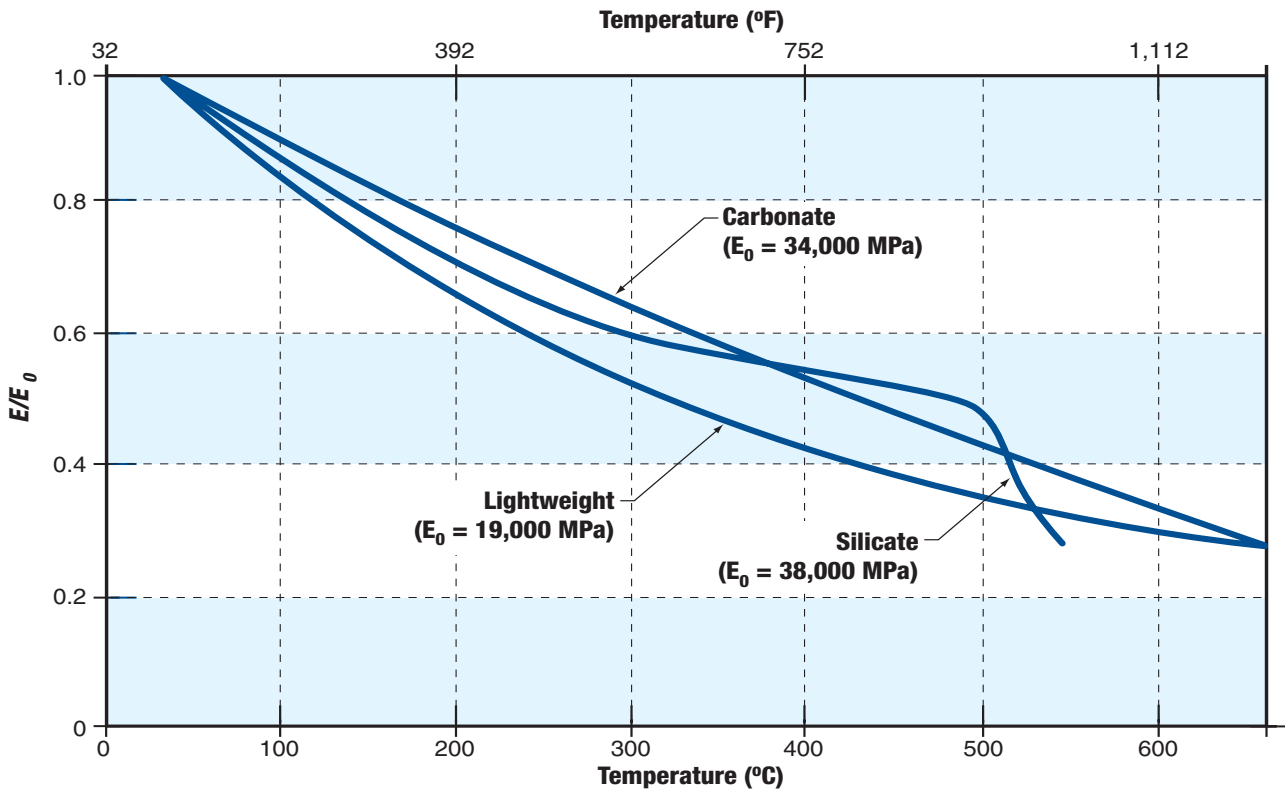


Figure A-11 The effect of temperature on the modulus of elasticity strength of different types of concretes (Kodur and Harmathy 2002).

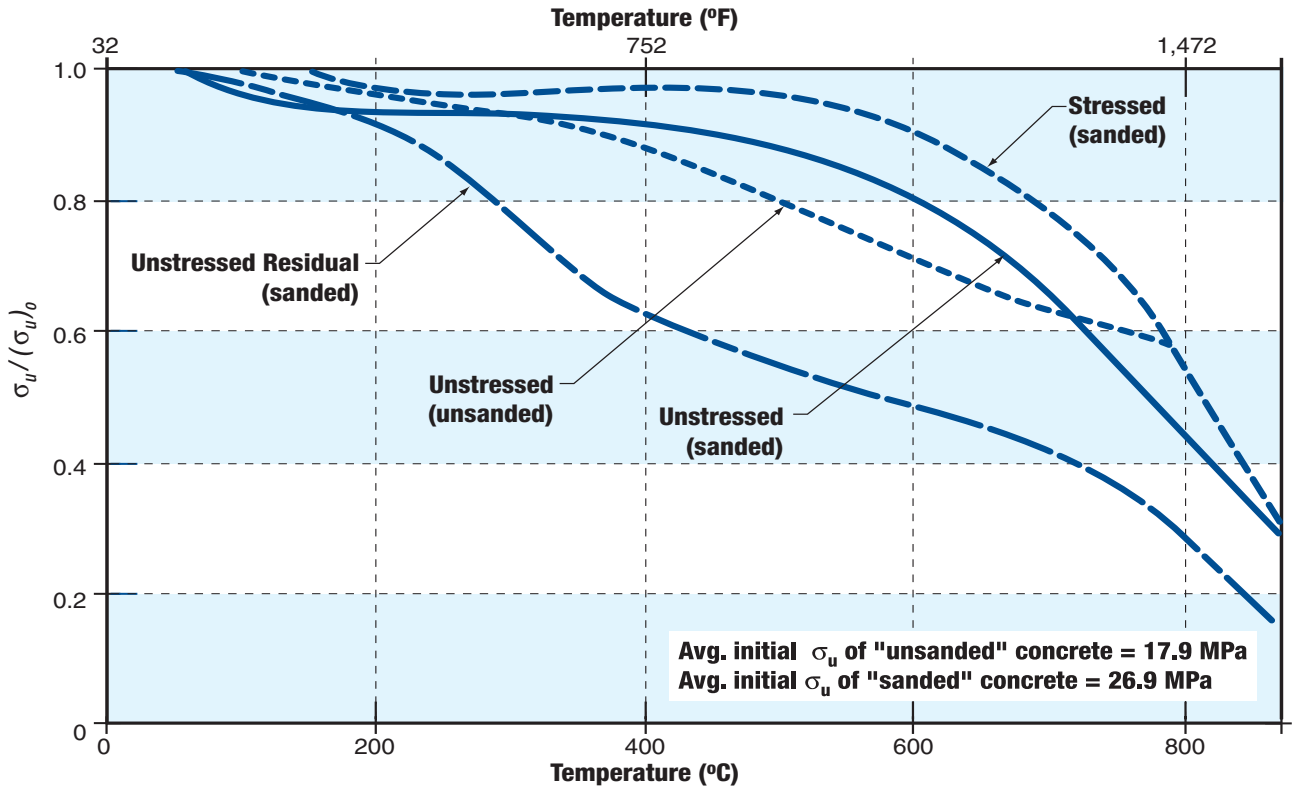


Figure A-12 Reduction of the compressive strength of two lightweight concretes (one with natural sand) at elevated temperatures (Kodur and Harmathy 2002).

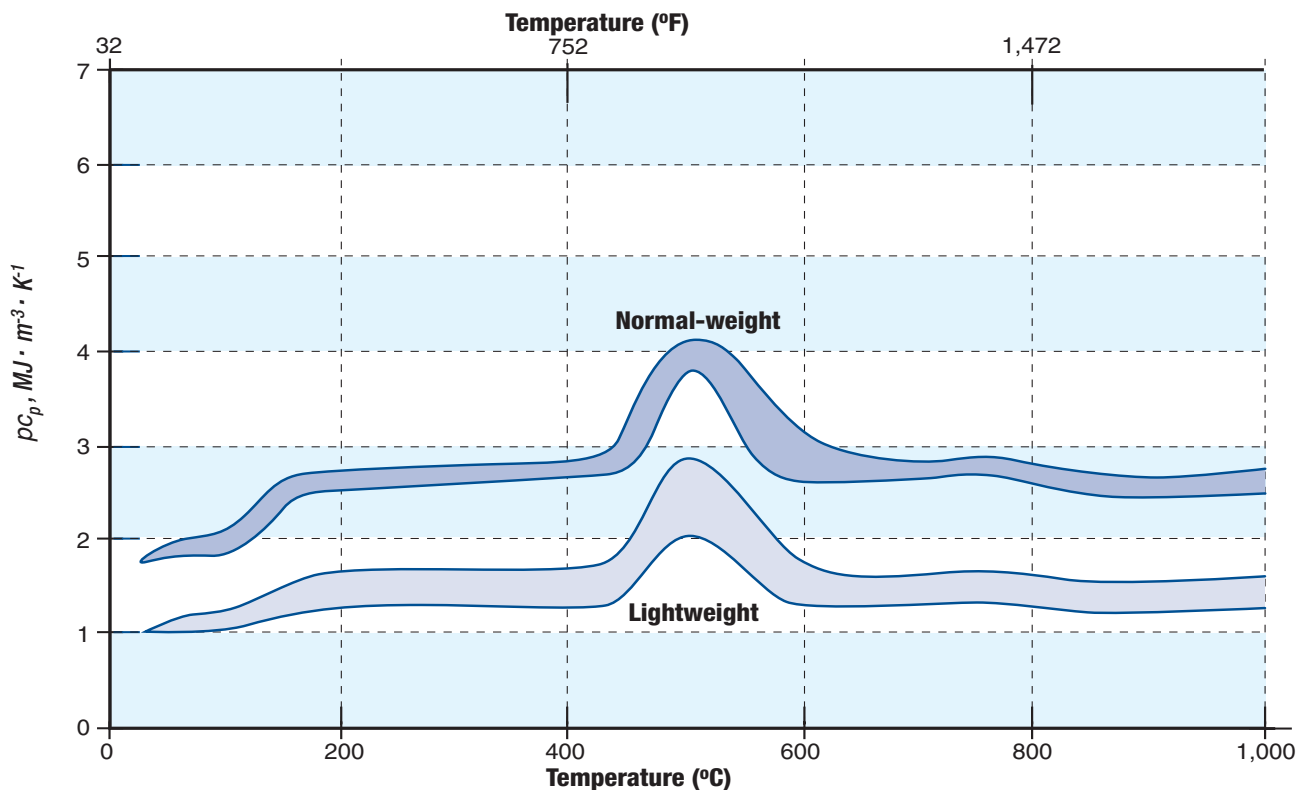


Figure A-13 Usual ranges of variation for the volume-specific heat of normal-weight and lightweight concretes (Kodur and Harmathy 2002).

Spalling is defined as the breaking of layers (pieces) of concrete from the surface of the concrete elements when it is exposed to high and rapidly rising temperatures. The spalling can occur soon after exposure to heat and can be accompanied by violent explosions, or it may happen when concrete has become so weak after heating that, when cracking develops, pieces fall off the surface. The consequences may be limited as long as the extent of the damage is small, but extensive spalling may lead to early loss of stability and integrity due to exposed reinforcement and penetration of partitions.

The extent of spalling is influenced by fire intensity, load intensity, strength and porosity of concrete mix, density, aggregate type, and internal moisture content of the concrete. Significant spalling can occur if the concrete has high moisture content and is exposed to a rapid growth fire.

### **A.3.3 Fire and Structural Modeling**

Fire protection provided in accordance with building codes is based on laboratory tests that have no correlation with actual fires. Through the use of numerical models, the fire protection design of structural members can be determined given the exposure conditions from selected fire scenarios.

Building code requirements for fire resistance design are currently based on the presumed duration of a standard fire as a direct function of fire load, building occupancy, height, and area. The severity of actual fires is determined by additional factors, which are not now considered in current building codes except as an alternate material method or equivalency when accepted by the enforcing official. Recent fire research provides a basis for designing fire protection for structural members by analytical methods and is becoming more acceptable to the building code community. In recent years, the use of numerical methods to calculate the fire resistance of various structural members has begun to gain acceptance. These calculation methods are reliable and cost-effective and can be applied to analyze performance in a specific situation (Milke 1999). The Eurocodes currently describe a calculation method for assessing the performance of steel members exposed to actual fires. There are three analyses that need to be conducted in a numerical assessment of fire resistance:

- model fire development
- model thermal response of assemblies
- model structural response of assemblies

Fire development is modeled to describe the heating exposure provided by the fire. Next, the thermal response analysis consists of predicting the temperature rise of structural members. Finally, an analysis of structural performance can be conducted to determine the structural integrity or load carrying capacity of the fire-exposed structural members. Such an analysis needs to account for thermally-induced deformations and property changes.

The analysis of the WTC buildings and the evaluation of other existing and future tall buildings could involve both fire and structural modeling. Both mathematical and scale modeling, along with validation tests, may be needed. In terms of the numerical modeling, it is currently possible to assemble a model package that reasonably predicts the impact of the fire on strength, elongation, spalling, and other properties related to the structural stability of the buildings involved. Currently, the available models for air movement (to the fire), fire growth and the resulting environmental condition in the space, breaking of windows, heat transfer through materials (e.g., fireproofing), and temperature rise in structural elements operate independently of each other and generally do not share data. In the future, combined fire-structural models may emerge that can interactively feed the output from heat transfer analysis models to structural analysis routines on a time basis as the simulated fire progresses, with return feed to the fire models of any changes

(pertinent to the fire model) that the structural computations predict, such as changes in ventilation characteristics. The combined fire-structural model(s) would permit extending the analysis of the impact of this incident to other scenarios, such as fire alone or other combinations of multiple simultaneous impacts (e.g., fire with wind, earthquake) on buildings.

Although the current models are based on sound physics, the state of the art of existing models involves uncertainties. Most of the models needed to supply the structural designer with case-specific data on temperatures of the exposed structural elements in unit area increments matching the finite elements selected for structural analysis exist. However, most of these models are as yet only partially validated.

#### A.4 Life Safety

The matter of high-rise evacuation has become preeminent in fire and building discussions since September 11, 2001, as a result of the fatalities of over 3,000 building occupants and emergency personnel. Life safety is provided to building occupants by either giving them the opportunity to evacuate or be protected in place. Basic life safety principles include notification, evacuation (including relocation to other floors), and protection in place (SFPE 2000).

**Notification:** Occupants need to be notified promptly of an emergency. In addition, communication systems should be provided that allow automatic messages to be transmitted to occupants to give them specific instructions on how to respond. These messages may also be delivered over public address systems by building safety managers or fire suppression personnel.

**Evacuation:** This aspect involves providing people with the means to exit the building. The egress system involves the following considerations:

- Capacity - A sufficient number of exits of adequate width to accommodate the building population need to be provided to allow occupants to evacuate safely.
- Access - Occupants need to be able to access an exit from wherever the fire is, and in sufficient time prior to the onset of untenable conditions. Alternative exits should be remotely located so that all exits are not simultaneously blocked by a single incident.
- Protected Escape Route - Exits need to be protected by fire-rated construction to limit the potential for fire and heat to impact these routes until the last occupant can reach a place of safety. In addition, such routes may also be smoke protected to limit smoke migration into the route.

In general, the means of egress system is designed so that occupants travel from the office space along access paths such as corridors or aisles until they reach the exit. An exit is commonly defined as a protected path of travel to the exit discharge (NFPA 101 2000). The stairways in a high-rise building commonly meet the definition of an exit. In general, the exit is intended to provide a continuous, unobstructed path to the exterior or to another area that is considered safe. Most codes require that exits discharge directly to the outside. Some codes, such as NFPA 101, permit up to half of the exits to discharge within the building, given that certain provisions are met.

Design considerations for high-rise buildings relative to these two options involve several aspects, including design of means of egress, the structure, and active fire protection systems, such as detection and alarm, suppression, and smoke management.

There is no universally accepted standard on emergency evacuation. Many local jurisdictions through their fire department public education programs have developed comprehensive and successful evacuation planning models, but unless locally adopted, there is no legal mandate to exercise the plans. Among the cities that have developed comprehensive programs are Seattle, Phoenix, Houston, and Portland, Oregon.

**Protect in Place:** The protect in place strategy is commonly employed in high-rise buildings. Occupants either remain in an area enclosed in fire rated construction or move to such a location. This approach is especially important for mobility impaired individuals. Building construction and fire protection systems are employed to protect occupants from fire and smoke spread for the duration of the incident or until rescued.

In some cases, occupants may be moved from one location to a location of relative safety while they await rescue. The Americans with Disabilities Act (ADA) of 1990 (42 USC 12181), in its design guidelines for new construction since 1993, requires that each floor in a building without a supervised sprinkler system must contain an “area of rescue assistance” (i.e., an area with direct access to an exit stairway where people unable to use stairs may await assistance during an emergency evacuation). In existing buildings, the ADA makes no reference to occupant evacuation other than to prohibit unnecessary physical barriers to mobility.

Additional information about courses and publications on emergency evacuation can be obtained at <http://www.usfa.fema.gov>.

#### A.4.1 Evacuation Process

Two methods are followed for the evacuation of buildings. One method consists of evacuating all occupants simultaneously. Alternatively, occupants may be evacuated in phases, where the floor levels closest to the fire are evacuated first, then other floor levels are evacuated on an as needed basis. Phased evacuation is instituted to permit people on the floor levels closest to the fire (i.e., those with the greatest hazard) to enter the stairway unobstructed by queues formed by people from all other floors also being in the stairway. Those who are below the emergency usually are encouraged to stay in place until the endangered people from above are already below this respective floor level. Generally, phased evacuation is followed in tall buildings, such as WTC 1 and WTC 2.

#### A.4.2 Analysis

A fairly simplistic model can be applied to develop a first order approximation of the time required to evacuate a high-rise building. The model is described by Nelson and MacLennan in the SFPE Handbook of Fire Protection Engineering. The following calculations are based on several major assumptions:

- All persons start to evacuate at the same time and hence no pre-movement time is considered (e. g., talking to coworkers, turning off computers, putting on coats).
- Occupant travel is not interrupted to make decisions or communicate with other individuals involved.
- The persons involved are free of any disabilities that would significantly impede their ability to keep up with the movement of the group. This includes any temporary disabilities as a result of fatigue.
- Firefighters coming into the stairway do not impose a significant impact on the flow rate of occupants traveling down the stairs.
- The controlling feature of the flow rate of people from the building is the door at the bottom of the exit stairway. This assumes that people develop a queue in the stairway that ends at the doorway at the base of the stairway. Also, the time for the first people to form the queue is assumed to be much less than the total evacuation time.
- The density of the people traveling through the doorway is in the range of observed values (i.e., 6-10 ft<sup>2</sup>/person). As such, the flow rate per foot of effective width for each doorway would be anticipated to be in the range of 18 to 24 persons/min (see Figure A-14). Consequently, the flow rate from each doorway in the World Trade Center buildings would have been on the order of 30 to 50 persons/min.



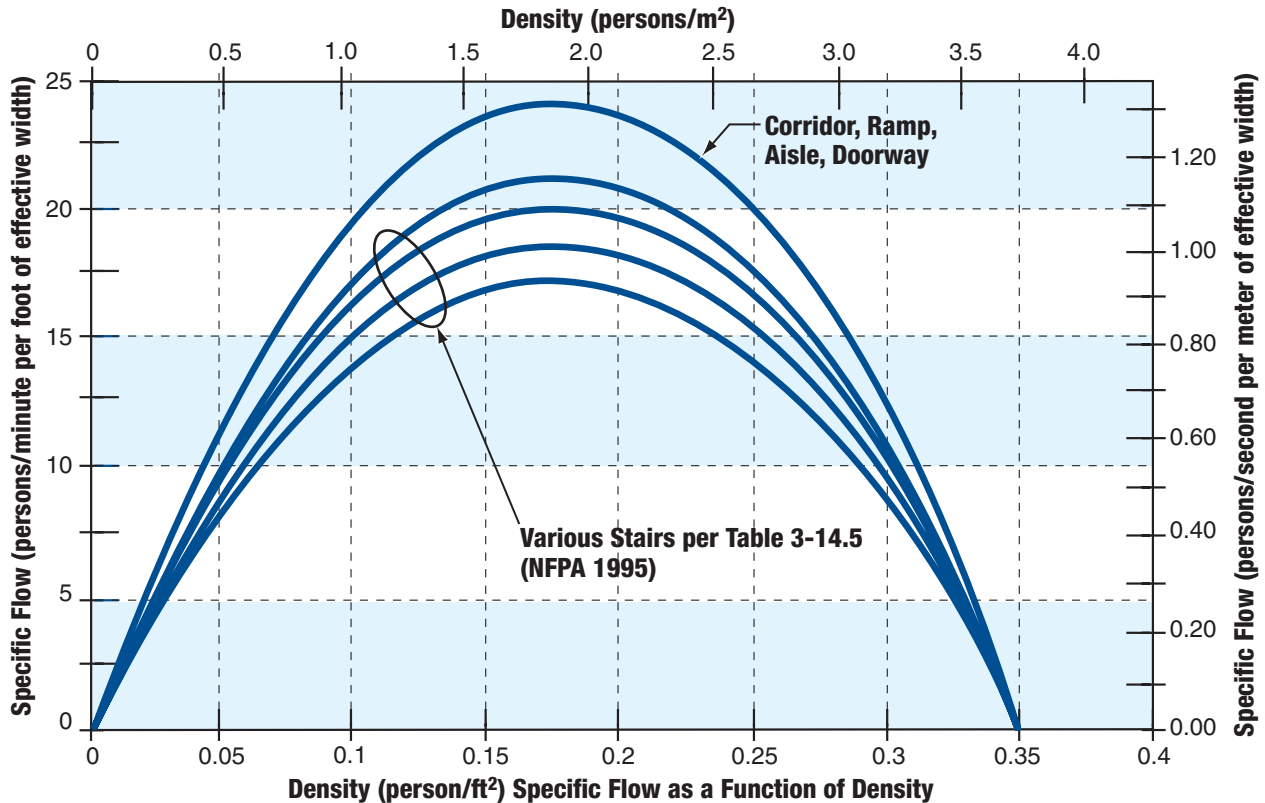


Figure A-14 Specific flow rate as a function of density (SFPE Handbook of Fire Protection Engineering).

Given these assumptions, the results presented in Figure A-15 relate to a lower limit of the time expected to evacuate the WTC towers. There were three exit stairways serving most floors of the WTC towers. Below the impact area, all stairways appeared to be available. The number of people in each building on the morning of September 11, 2001, is not known. Therefore, a range of occupant loads is included in Figure A-15.

By all indications, it was instantly apparent to the building occupants that evacuation was necessary, so very little time was likely to have transpired in pre-movement activities. The time for the leading edge of the evacuees to reach the stairs and to descend from the lowest occupied floor (7) to the discharge doors on floors 1 and 2 is estimated to have taken about 3 minutes until the steady human flow reached its capacity. The sense of urgency in the evacuees is estimated to have maintained the egress flow at or near the theoretical maximum for stair exit flow (i.e., 24 persons/minute per foot).

The two end stairs were 44 inches wide and the center stair was 56 inches wide. Each stair had a single 36-inch-wide exit door at its discharge level. As such, the effective width for each stair door was 24 inches (2 feet). The expected steady flow rate from the stair doorways was 48 persons/minute. Based on an available egress time of 90 minutes in WTC 1 and 50 minutes in WTC 2, the number of persons who could have exited through the stairs is estimated to be up to 13,000 for WTC 1 and up to 7,200 for WTC 2. These estimates do not include any persons who used elevators, were on the 2nd (Plaza) level or lower in the buildings at the time, or initiated evacuation in WTC 2 immediately after the impact of WTC 1.

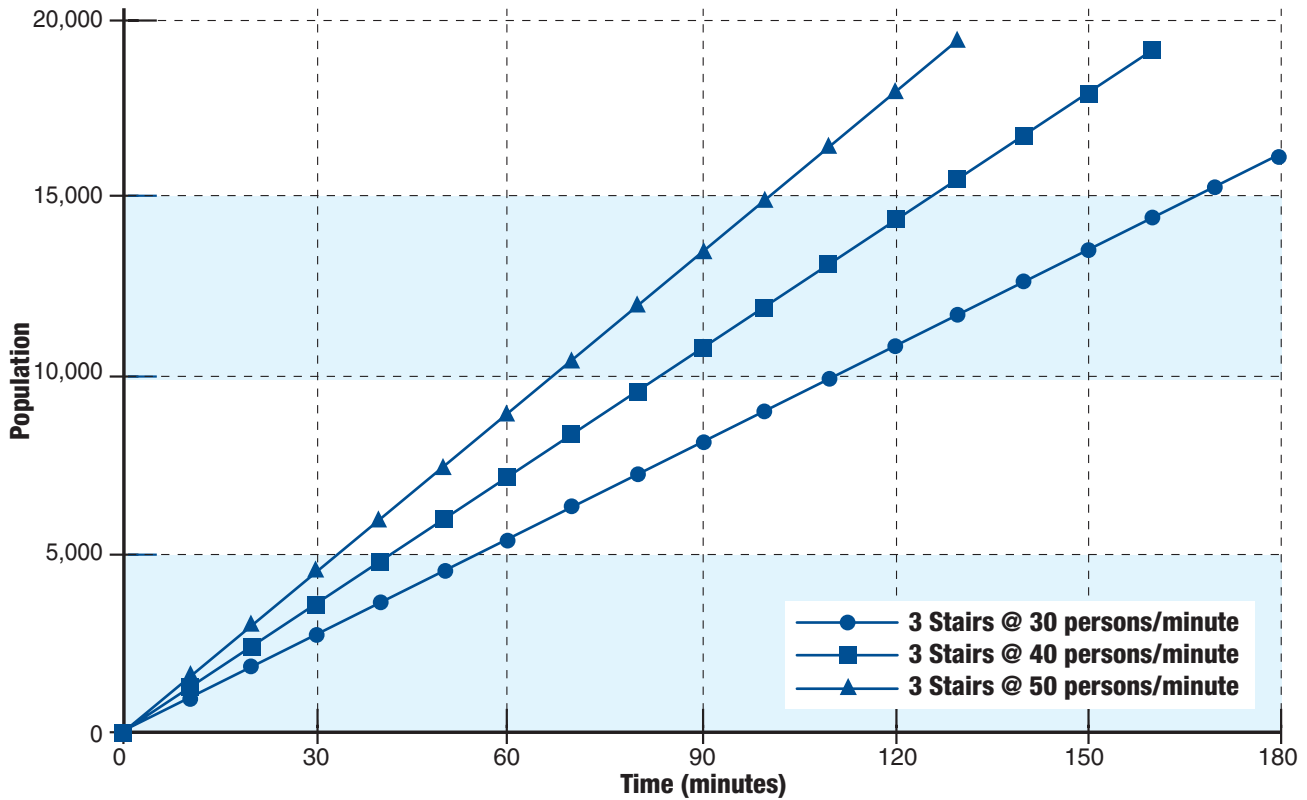


Figure A-15 Estimated evacuation times for high-rise buildings.

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