4. Joists and Trusses – Seven thickness measurements should be made at each end of a 12 in. (305 mm) length in a pattern as shown in Figure IX.3.
5. Columns – Twelve thickness measurements should be made at each end of a 12 in. (305 mm) length in a pattern as shown in Figure IX.4.

Minimum Allowable Thickness: The average calculated thickness of the SFRM is to be greater than or equal to the design thickness. Section 1704.11.3 of the IBC requires individual measurements for the design thickness of 1 in. (25.4 mm) or greater to be no less than the design thickness minus ¼ in. (6.4 mm). For the design thickness of less than 1 in. (25.4 mm), the IBC limits the individual measurement to be no less than the design thickness minus 25 percent.

Procedure in case of deficiency: ASTM E605 requires deficient items to be corrected and retested along with another item of the same type selected at random. When an item does not meet the prescribed requirements, only that specific element is deemed deficient. All other items in the bay, as well as similar elements in other areas of the building, are not to be considered deficient based solely on the failure of the tested item.

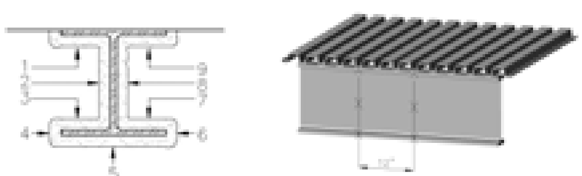

Fig. IX.2 SFRM Thickness Measurement Locations at Beams

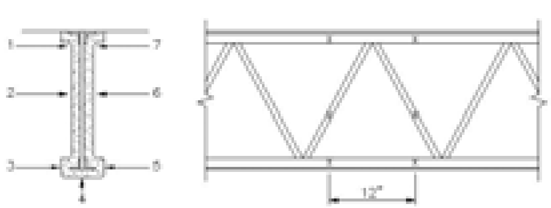

Fig. IX.3 SFRM Thickness Measurement Locations at Joists

SFRM thickness deficiencies on an element may be corrected by simply applying additional material. As an alternative to this method, certain fire resistance rating criteria and some SFRM manufacturers publish thickness to density correction formulas. These formulas may allow an item found lacking in thickness, but exceeding the requirements for density, or vice versa (i.e. the overall weight in pounds per square foot is equivalent), to be considered passing. The testing agency that published the fire assembly should be consulted when using this procedure.

IX.3 DENSITY DETERMINATION ASTM E605

Procedure: This procedure is valid for both sprayed fiber and cementitious types of SFRM. The density of the sample is determined by first scoring the specimen around the perimeter of a rectangular template. A minimum of 12 thickness measurements should be taken symmetrically within the scored region and averaged. The specimen should then be cut away from the substrate along the perimeter of the template and removed.

The density of the specimen is then calculated as follows:

$$D = \frac{W}{l * w * t} \qquad \textbf{(IX-1)}$$

where
D = density, lb/ft^3 (kg/m^3)
W = constant weight of dried material, lb (kg)
l = length of specimen, ft (m)
w = width of specimen, ft (m)
t = average thickness of the field measurements of the specimen, ft (m)

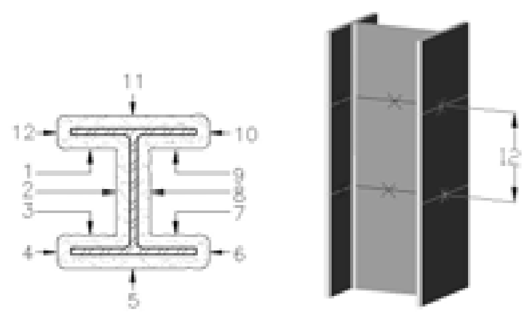

Fig. IX.4 SFRM Thickness Measurement Locations at Columns

ASTM E605 also lists an alternate, displacement-type method for determining the SFRM density applied to irregular surfaces or for specimens that are difficult to remove from the substrate.

Testing frequency: Tests should be taken at every floor or per every 10,000 ft^2 (929 m^2), whichever provides the greater number of tests.

Member tests: ASTM E605 requires density tests to be taken at random for each of the following elements: the flat portion of the deck; a beam, either the bottom of the beam lower flange or the beam web; and a column, either the column web or the outside of one of the column flanges.

Minimum Allowable Density: The average and individual SFRM density measurements are to meet the manufacturer's minimum specifications for the designated mix.

Procedure in case of deficiency: If an item is found to be deficient, the same procedure as described in Section IX.2 Thickness Determination should be followed.

IX.4 COHESION/ADHESION DETERMINATION ASTM E736

Procedure: This procedure is valid for both sprayed fiber and cementitious types of SFRM. A metal or rigid plastic bottle screw cap with attached hook, as shown in Figure IX.5, is filled with urethane resin adhesive and immediately placed against the surface of the SFRM. All excess adhesive around the edges of the cap should be removed. A spring-type weighing scale is then attached to the hook. A force of 11 lb (5 kg) per minute, applied either at a minimum uniform or incremental rate, is engaged to the scale, perpendicular to the SFRM surface. The force shall be applied until either a predetermined value is achieved or failure occurs. As an alternate, a non-destructive field test may be performed by supporting a fixed weight for 1 minute.

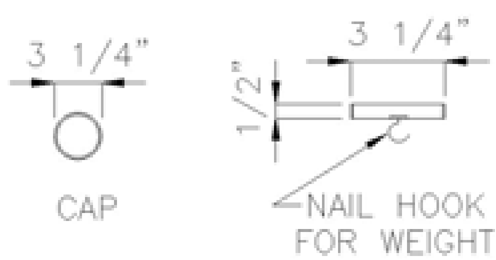

Fig IX.5 Test Cap[1]

The cohesive/adhesive force is then calculated as follows:

$$CA = \frac{F}{A} \quad \textbf{(IX–2)}$$

where

CA = cohesive/adhesive force, lb/ft^2 (N/m^2)
F = recorded force, lb (N)
A = area of the cap, ft^2 (m^2)

Testing frequency: The IBC requires the number of tests for members as follows:

> *1704.11.5.1 Floor, roof and wall assemblies. The test samples for determining the cohesive/ adhesive bond strength of the sprayed fire-resistant materials shall be selected from each floor, roof and wall assembly at the rate of not less than one sample for every 10,000 square feet (929 m^2) or part thereof of the sprayed area in each story.*
>
> *1704.11.5.2 Structural framing members. The test samples for determining the cohesive/ adhesive bond strength of the sprayed fire-resistant materials shall be selected from beams, girders, joists, trusses and columns at the rate of not less than one sample for each type of structural framing member for each 10,000 (929 m^2) square feet of floor area or part thereof in each story.*

Member tests: All tests should be performed on a minimum 12 in. by 12 in. (305 mm by 305 mm) area on the element. On members where this area is unavailable, such as beams and fluted decks, an area the size of the width of the beam or the flute by 12 in. (305 mm) is to be used. The minimum area shall not be less than 4 in. by 12 in. (102 mm by 305 mm).

Minimum Allowable Bond Strength: The IBC requires the minimum cohesive/adhesive bond strength of the cured specimen to not be less than 150 lb/ft^2 (7.2 kPa).

Procedure in case of deficiency: Certain fire resistance rating criteria and some SFRM manufacturers allow bonding agents or mechanical attachments to be used where bond strength test results are found to be less than the minimum accepted values. The SFRM manufacturer should be consulted.

REFERENCES

[1] The Association of the Wall and Ceiling Industries, International (AWCI) (1997), *Technical Manual 12 A, Standard Practice for the Testing and Inspection of Field Applied Sprayed Fire-Resistive Materials; an Annotated Guide*, Third Edition, Falls Church, VA.

[2] American Society for Testing and Materials (ASTM) (2000), *Standard Test Methods for Thickness and Density of Sprayed Fire-Resistive Material (SFRM) Applied to Structural Members*, Specification No. E605-93, West Conshohocken, PA.

[3] American Society for Testing and Materials (ASTM) (2000), *Standard Test Method for Cohesion/Adhesion of Sprayed Fire-Resistive Materials Applied to Structural Members*, Specification No. E736-00, West Conshohocken, PA.

[4] Underwriters Laboratories Inc. (UL) (2003), *Fire Resistance Directory, 2003*, Vol. 1, Northbrook, IL.

[5] Southern Building Code Congress International (SBCCI) (1997), *1997 Standard Building Code*, Birmingham, AL.

[6] International Conference of Building Officials (ICBO) (1997), *1997 Uniform Building Code*, Volume 1, Whittier, CA.

[7] International Code Council, Inc. (ICC) (2000), *International Building Code, 2000,* Falls Church, VA.

Section X ENGINEERED FIRE PROTECTION

X.1 GENERAL INFORMATION

The primary objective of a fire protection system is to allow the structure to function (i.e. sustain load and limit spread of the fire) for a sufficient time to permit occupant egress from the facility, fire suppression operations and search and rescue operations. Analytical tools are available to simulate the heat generated by combustion of the building contents. The heat determined by analysis can be used as input to a generalized equation for temperature rise in a protected or unprotected steel section. Thus, the temperature gradient of a steel member caused by the heat input can be predicted at a specific time. All materials degrade with increases in temperature. The relationship between specific steel properties and temperature are well documented. It is possible to predict the effective yield strength at an elevated temperature and, using the appropriate value, the capacity of the steel section at the elevated temperature can be determined.

X.2 BUILDING CODES

The most frequently employed building code provisions relating to fire protection are prescriptive. For the case of roof and/or floor construction, the required level of fire resistance is determined from the building code. Then an assembly that has been tested for fire endurance is referenced and adjustments to the thickness of fire protection materials are determined so that the actual beam performance will approximate the tested beam performance.

The building code does permit the use of an alternate approach. Alternative approaches may be done on a case by cases basis, or the entire building may be designed using a performance based design approach.

NFPA 5000, *Building Construction and Safety Code*[1], establishes an equivalency approach in Section 1.5. Numerous criteria exist for approval of an equivalent design method and all such designs must be acceptable to the authority having jurisdiction. The charging language for equivalency states:

> *1.5.1 General. Nothing in this Code shall be intended to prevent the use of systems, fire resistance, effectiveness, durability, and safety over those prescribed by this Code. Technical documentation shall be submitted to the authority having jurisdiction to demonstrate equivalency. The system, method, or device shall be approved for the intended purpose by the authority having jurisdiction (AHJ).*

Conversely, NFPA 5000, Section 4.3, permits the use of performance based design approaches that comply with Chapter 5 of the Code to be applied to the entire design process.

> *4.3.1 Options. Building design meeting the goals and objectives of Section 4.1 shall be provided in accordance with either of the following:*
>
> *(1) The prescriptive-based provisions of 4.3.2*
> *(2) The performance-based provisions of 4.3.3*

Regardless of whether the design is done using the purely prescriptive approach, an equivalency approach for parts of the design, or if it is done using the performance based design approach, the specified goals and objectives contained within Chapter 4 of NFPA 5000 must still be adhered to.

Among these goals is the statement in Section 4.4.8 that requires an appropriate level of protection for the structural system. That goal is noted under the heading dealing with multiple and appropriate safeguards. It states:

> *4.4.8 Structural Integrity. The building's structural members and assemblies shall be provided with the appropriate degree of fire resistance to limit structural damage to an acceptable level and to limit damage to the building and its contents and to adjacent buildings and property.*

The International Building Code (IBC)[2] permits the use of alternate approaches to the prescriptive procedures in paragraph 104.11.

> *104.11 Alternative materials, design and methods of construction and equipment. The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, at least the equivalent of*

that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety.

According to both codes, subject to the approval of the authority having jurisdiction (AHU), where calculations and/or other documentation confirm the ability of the structural system to remain functional for the intended time period under an expected fire exposure, an equivalent level of protection can be verified.

X.3 LOAD COMBINATIONS

A fire is recognized as a low-probability event in paragraph 2.5 of ASCE 7[3]. The load combinations to be used in checking the capacity of a structure or structural element to withstand the effects of an extraordinary event are presented in the ASCE 7 commentary paragraph C2.5. There are two load combinations to be considered as follows:

$$1.2D + A_k + (0.5\,L \text{ or } 0.2S) \qquad \textbf{(X – 1)}$$

$$(0.9 \text{ or } 1.2)D + A_k + 0.2W \qquad \textbf{(X – 2)}$$

where

A_k = load effect from the extraordinary event
D = dead load
L = live load
S = snow load
W = wind load

The load effect of the fire extraordinary event has a load factor of 1.0 and the companion actions of $0.5L$, $0.2S$ and $0.2W$ reflect the small probability of joint occurrence of all loads.

X.4 HEAT TRANSFER

An all inclusive heat transfer evaluation involves a complex three-dimensional analysis including the influence of radiation and convection. In addition to the typical conductive heat transfer, the mechanism involves convection and radiation between the surface of the protection and the fire, as depicted in Figure X.1. Furthermore, most protective materials have some moisture content, and the heat required to vaporize the moisture affects the rate of temperature increase in the steel. However, a simplification using a one-dimensional heat transfer equation can generally be used to predict steel temperature increases with reasonable accuracy.

A generalized equation[4] for the temperature rise in a protected steel section considering the thermal capacity of the insulation is:

$$\Delta T_s = \frac{k_p}{d_p}\left[\frac{T_f - T_s}{c_s\dfrac{W}{D} + \dfrac{c_p\,\rho_p\,d_p}{2}}\right]\Delta t \qquad \textbf{(X – 3)}$$

where

ΔT_s = change in steel temperature in time step Δt (°C)
Δt = time step (sec.)
T_s = steel temperature (°C)
T_f = fire (furnace) temperature (°C)
D = heated perimeter (m)
W = weight per unit length (kg/m)
c_s = specific heat of steel (J/kg °C)
c_p = specific heat of insulation (J/kg °C)
ρ_p = density of insulation (kg/m^3)
d_p = thickness of insulation (m)

Note: The *W/D* ratio in Equation X-3 has units of kg/m^2. The *W/D* ratios listed by AISC and reproduced in Appendix A have units of lb/ft-in. To convert the *W/D* ratios listed in Appendix A to SI units multiply by 58.6.

Thus, Equation X-3 defines the increase in temperature of an insulated steel section as a function of the thermal properties of the steel and insulation assuming a one-dimensional heat transfer equation.

The specific heat values for steel and spray-applied fire resistive materials (SFRM) vary with temperature. However, using a constant value for these parameters results in a reasonably accurate correlation between calculations and test results. The specific heat of steel may be evaluated at 572 °F (300 °C) and the specific heat of SFRM may be considered at 932 °F (500 °C).

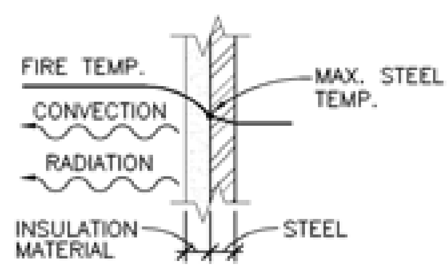

Fig. X.1 Heat Transfer Mechanism within an Insulated Steel Member

A representative value for the specific heat of steel is:

$$c_s \approx 560 \text{ J/kg°C } (0.13 \text{ Btu/lb °F})$$

The thermal properties of SFRM vary between manufacturers. Representative values for lightweight SFRM are:

$$\rho_i = 293 \text{kg/m}^3 \ (18.30 \text{ pcf})$$

$$C_i = 754 \text{ J/kg°C } (0.18 \text{ Btu/lb °F})$$

$$k_i = 0.135 \text{ W/m°C } (0.0013 \text{ Btu/hr ft °F})$$

The standard time temperature curve defined in ASTM E 119[5] represents the heat input for the tested assemblies listed in the Underwriters Laboratory (UL) Directory[6]. Thus, the time-temperature relationship required by ASTM E119 can be used as input to Equation X-3 and the calculated temperature compared to the measured temperature to confirm the accuracy of Equation X-3.

A spreadsheet can be created to use time step increments and predict the temperature increase in the protected steel beam. The representative thermal values of insulation have been used with the heat input from a standard fire to generate time temperature curves. A time step increment of one second was used in these numerical predictions. The calculated results were found to closely match measured bottom flange temperatures.

A representative comparison is presented graphically in Figure X.6. At 120 minutes, the calculated temperature of the bottom flange of the tested W8x28 with 1 in. (25.4 mm) of SRFM is 1,240 °F (671 °C) and the measured temperature was 1,200 °F (649 °C).

X.5 TEMPERATURE GRADIENT

A fire beneath a structural floor initially affects the bottom flange, and a temperature gradient develops over the depth of the beam member. A concrete slab over the top flange acts as a heat sink, dissipating heat away from the steel. This heat dissipation can result in a substantial difference in temperature between the bottom and top flange.

Review of numerous test results conducted by UL confirmed that it is reasonable to assume a 25 percent reduction in the top flange temperature from that of the bottom flange. The temperature gradient is not linear between the top flange and the bottom flange; the mid-depth temperature is generally higher than the average temperature between the flanges. Thus, it is conservative to assume a mid-depth temperature equal to the bottom flange temperature and a linear decrease in temperature from mid-depth to the top of the top flange. This gradient is depicted in Figure X.2.

X.6 STEEL PROPERTIES AT ELEVATED TEMPERATURES

Steel properties, like those of other conventional construction materials, degrade with increases in temperature. The influence of elevated temperatures on the modulus of elasticity (E_m) and the yield strength (F_{ym}) of steel is presented in Table X.1 as a ratio of the value at the elevated temperature to the value at 68 °F (20 °C)[7].

X.7 COMPOSITE STEEL BEAM CAPACITY AT ELEVATED TEMPERATURES

As mentioned previously, steel properties diminish when subjected to elevated temperatures. However, as fire conditions are considered to represent an ultimate state for the element, and serviceability is not an issue, the designer may fully utilize the plastic capacity and moment redistribution effects within typical composite sections. Additionally, reduced load factors, as mentioned above, may also be applied to the beam due to the extraordinary nature of a building fire and the unlikely probability of the full design load occurring during the event.

Under fire conditions, the rotational restraint provided by the continuous floor slab allows the moment diagram to shift, considering negative moment demands at the beam ends. Capacity for this demand at the ends of the beam is provided by the slab reinforcing steel running parallel with the beam coupled with the steel section[8]. Therefore, the decrease in the positive capacity of the beam section during the event is compensated as the moment demand shifts into the negative bending reserve provided at the beam ends. The beam section retains

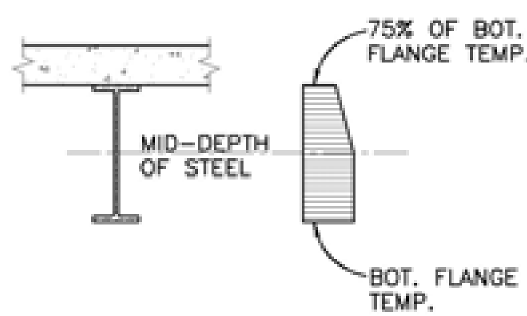

Fig X.2 Illustration of Design Temperature Gradient

Table X.1 Steel Modulus of Elasticity and Yield Strength Reduction at Elevated Temperatures		
Steel Temperature °F [°C]	E_m/E	F_{ym}/F_y
68 [20]	1.00	1.00
200 [93]	1.00	1.00
400 [204]	0.90	1.00
600 [316]	0.78	1.00
750 [399]	0.70	1.00
800 [427]	0.67	0.94
1,000 [538]	0.49	0.66
1,200 [649]	0.22	0.35
1,400 [760]	0.11	0.16
1,600 [871]	0.07	0.07
1,800 [982]	0.05	0.04
2,000 [1,090]	0.02	0.02
2,200 [1,200]	0.00	0.00

the ability to successfully support its load as long as the moment demand falls within this envelope of positive and negative moment capacities, as shown in Figure X.3.

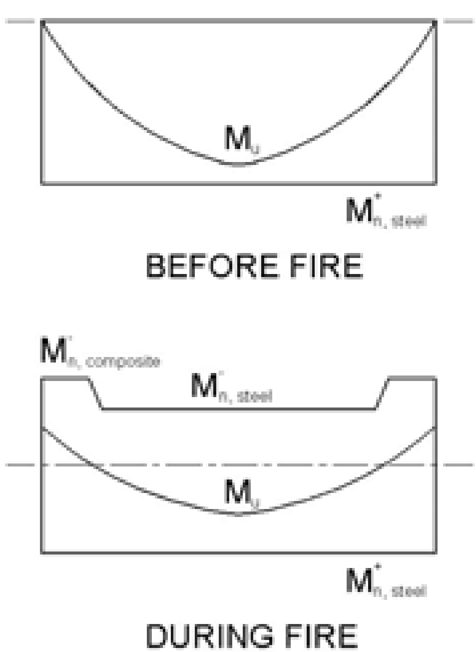

Fig X.3 Moment Envelops Before and During Fire Event[8]

X.7.1 Positive Nominal Flexural Strength. Once the temperature gradient is known across the beam section (Figure X.2), and its corresponding effects upon the physical properties of the beam are known (Table X.1), the positive nominal flexural strength at mid-span can be determined.

It is often convenient to begin with the assumption that the neutral axis lies within the concrete slab. Under this assumption, the concrete slab is in the compression zone, and the entire steel section provides the tensile component. The concrete is modeled to achieve its full plastic capacity at a strain of 0.003, and the steel is assumed to be fully yielded. The assumption of designing the concrete to its full plastic capacity is generally valid as the temperature of the top surface of the slab does not increase dramatically. This is a reasonable assumption, given the limit on temperature rise for the unexposed side of 250 °F (121 °C) in standard fire resistance tests.

With these assumptions, and the reduced steel yield strengths along the section of the beam, the tensile component of the flexural capacity, F_T, may be determined.

$$F_T = F_{tf} + F_w + F_{bf} \qquad \textbf{(X - 4)}$$

where

F_{tf} = yield capacity of the top flange at its elevated temperature (ksi or MPa)

F_w = yield capacity of the web at its elevated temperature (ksi or MPa)

F_{bf} = yield capacity of the bottom flange at its elevated temperature (ksi or MPa)

The depth of the equivalent rectangular compression block, a, may then be computed.

$$a = \frac{F_T}{0.85 * f'_c * b_f} \qquad \textbf{(X – 5)}$$

where

f'_c = compressive strength of concrete (ksi or MPa)

b_f = effective concrete slab width (in. or mm)

If the equivalent compression block is less than the slab depth, then the assumption that the neutral axis lies within the slab is valid. If not, then the compression in the steel can be accounted for as illustrated in the AISC Manual procedures for this case.

The nominal moment capacity, M_n, may then be determined by summing the moments of concrete and steel components about the neutral axis of the member. If the neutral axis lies within the slab, each element of the tensile resistance (F_{tf}, F_w, F_{bf}) forms a couple with a portion of the concrete compressive force (F_c). Therefore, the nominal moment capacity may be determined by multiplying each of these elements with its respective lever arm to the compressive force. A diagram of these forces is shown in Figure X.4.

X.7.2 Negative Nominal Flexural Strength. The negative nominal flexural strength at the ends of the beam is provided by the concrete slab reinforcement running parallel with the beam, possibly in combination with the top flange, acting in tension and forming a couple with the remaining portion of the steel beam acting in compression.

When computing the negative flexural strength, it is convenient to start with the assumption that the neutral axis is located within the beam web. It is also assumed that the steel is fully yielded in tension and compression, thus achieving its full plastic capacity. Again, a building fire represents an ultimate state, and serviceability requirements need not be satisfied.

With these assumptions, and the reduced steel yield strength values along the section of the beam, the initial iteration for the tensile component of the flexural capacity, F_T, may be found.

$$F_T = F_{RB} + F_{tf} \qquad \textbf{(X - 6)}$$

where

F_{RB} = tensile yield force of the reinforcing steel (ksi or MPa)

F_{tf} = yield capacity of the top flange at its elevated temperature (ksi or MPa)

The initial iteration for the compressive component of the flexural capacity, F_C, may then be run.

$$F_C = F_w + F_{bf} \qquad \textbf{(X - 7)}$$

where

F_w = yield capacity of the web at its elevated temperature (ksi or MPa)

F_{bf} = yield capacity of the bottom flange at its elevated temperature (ksi or MPa)

The above steps are then repeated until a state of equilibrium exists where

$$F_T = F_C \qquad \textbf{(X - 8)}$$

A diagram of these forces is shown in Figure X.5.

X.8 ANALYTICAL SFRM THICKNESS CALCULATION SUMMARY

Due to the iterative nature of the analytical approach to determining required amount of fire protection, the use of a spreadsheet is recommended. A summary for the approach is described below.

1. Using the Extraordinary Loading Condition (e.g. fire) of ASCE 7 (Equations X-1 and X-2), calculate the moment capacity required.

2. Choose a trial thickness of SFRM to be applied to the beam (minimum of ⅜ in. or 9.5 mm). It is often convenient to start with a thickness recommended by a UL assembly representative of the actual construction (e.g. UL D925, UL D902, etc.).

3. Using the heat transfer equation of thermal dynamics and heat input predicted by the standard fire test (ASTM E119) for the fire endurance required, calculate the steel temperature of the bottom flange at the required time rating, as shown in Equation X-3.

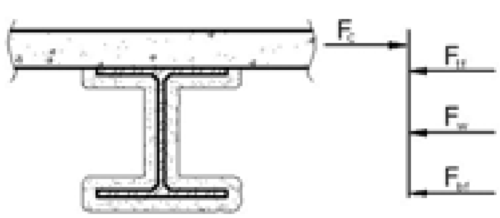

Fig X.4 Typical Positive Moment Force Resultants[8]

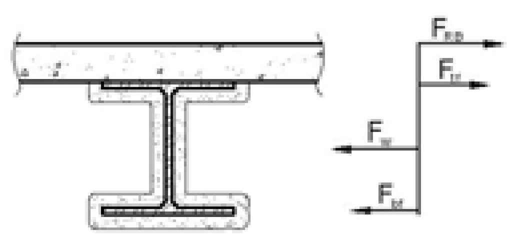

Fig X.5 Typical Negative Moment Force Resultants[8]

4. Calculate the steel temperature gradient across the remainder of the section, with the temperature staying constant from the bottom flange to the midpoint of the web, then linearly decreasing by 25 percent at the top flange, as depicted in Figure X.2.

5. Calculate the steel stress capacities available at the elevated temperatures for the beam bottom flange, web, and top flange using the values shown in Table X.1.

6. Determine the location of the plastic neutral axis and moment capacity at mid span using the bottom flange, web, and top flange forces available at elevated temperatures in conjunction with the compressive force in the concrete slab (composite action) as described in Section X.7.1, Positive Nominal Flexural Strength.

7. Determine the location of the plastic neutral axis and moment capacity at the supports using the bottom flange, web, and top flange forces available at elevated temperatures in conjunction with the tensile force in the reinforcing steel over the support as described in Section X.7.2, Negative Nominal Flexural Strength.

8. Confirm that the moment capacity envelope determined in 6 and 7 satisfy the capacity requirements as determined in 1. If the capacity is less than required, consider one of the following options:

   1. Increase the thickness of fireproofing.
   2. Increase the area of the reinforcing steel running parallel to the beam.
   3. Increase the steel beam size

X.9 ADVANCED METHODS OF ANALYSIS

Computer assisted numerical modeling has provided an exciting resource for investigating the performance of steel-framed structures subjected to fire.

To generate an accurate prediction of structural action in a fire, both the thermal input and the mechanical response must be considered. A realistic model for the thermal input is dependent upon the size of the fire compartment, the ventilation available throughout combustion and the type and quantity of combustibles involved. These parameters can be determined as a function of architectural layout (compartment size, ventilation openings) and occupancy (amount of combustibles). Thus, the thermal input can be rationally established as input to the established heat transfer and structural analysis models.

Structural response under fire conditions has been the focus of numerous investigations[4,9,10]. Computer models have been created that reflect the complex mechanisms of membrane action, thermally induced thrust, heat induced yielding, etc. The future of Fire Engineering will evolve from these pursuits. The result is certain to be the construction of more rational fire resistant steel structures and greater confidence in the fire safety of those built environments.

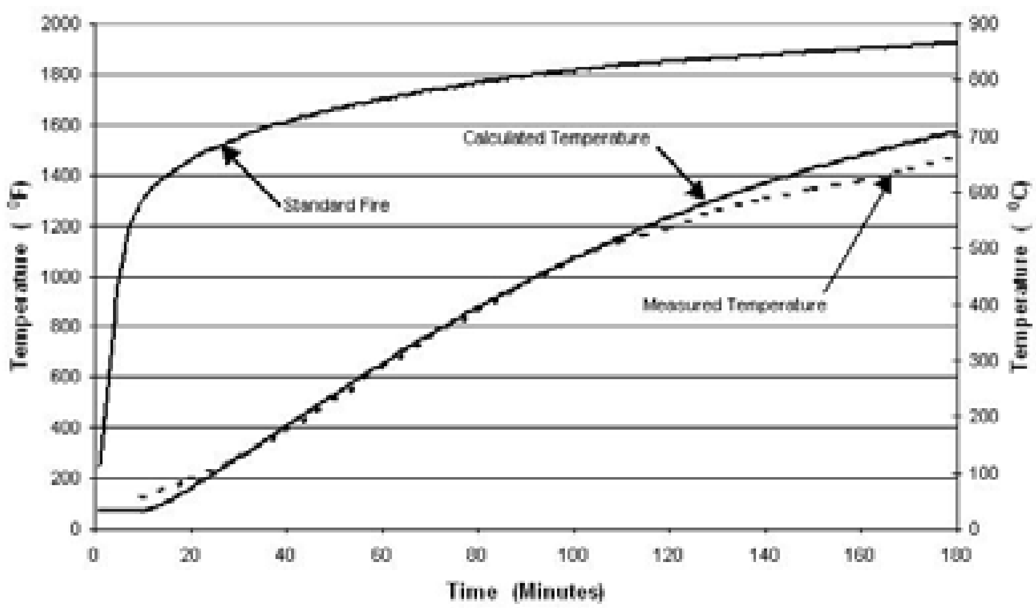

Fig X.6 Bottom Flange Temperature of W8x28 with 1in. SFRM Insulation

REFERENCES

[1] National Fire Protection Association (NFPA) (2003), *NFPA 5000: Building Construction and Safety Code,* 2003 Edition, Quincy, MA.

[2] International Code Council, Inc. (ICC) (2000), *International Building Code, 2000,* Falls Church, VA.

[3] SEI/ASCE 7-02 (2002), *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers (ASCE), Reston, VA.

[4] Milke, J.A. (2002), "Analytical Methods for Determining Fire Resistance of Steel Members", *The SFPE Handbook of Fire Protection Engineering*, Third Edition, National Fire Protection Association, Quincy, MA.

[5] American Society for Testing and Materials (ASTM) (2000), *Standard Test Methods for Fire Tests of Building Construction and Materials*, Specification No. E119-00, West Conshohocken, PA.

[6] Underwriters Laboratories Inc. (UL) (2003), *Fire Resistance Directory, 2003*, Vol. 1, Northbrook, IL.

[7] European Convention for Constructional Steelwork (ECCS) – Technical Committee 3 (2001), *Model Code on Fire Engineering,* First Edition, Brussels, Belgium.

[8] Ioannides, S.A., Mehta, S. (1997), "Restrained vs. Unrestrained Fire Ratings: A Practical Approach," *Modern Steel Construction*, May.

[9] Lie, T.T. (ed.) (1992), *Structural Fire Protection*, American Society of Civil Engineers, Reston, VA.

[10] Buchanan, A. H. (2001), *Structural Design for Fire Safety*, John Wiley & Sons, UK.