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Performance-Based Design of Structural Steel for Fire Conditions - A Calculation Methodology

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Abstract

Currently the designers of buildings in Canada rely on the results of standard fire tests to ensure building structures meet the fire resistance rating requirements prescribed by national building codes. With the development of performance-based building codes throughout America, It is important that the design community have the tools necessary to take advantage of these new codes. In order to provide structural engineers with these tools, a method is being proposed that will facilitate the design of structural steel for fire conditions using a performance-based approach. This approach is simplistic in nature, and only considers a two-dimensional thermal response of structural steel to the fire. A method has been proposed that allows the designer to predict the time-temperature relationship expected in a compartment fire with a reasonable level of conservatism, which can be used to determine the required level of protection.

Keywords: structural steel, fire response, fire resistance rating, fire scenario, performance-based design

1. Introduction

Under the current prescriptive code regime, there is generally no requirement to undertake an engineering approach to structural fire safety, since the required fire resistance ratings are prescribed and the fire resistance ratings of materials/assemblies are determined through standard tests. However, there is growing criticism that these standard tests may not be relevant based on current construction practices and materials, and that they do not accurately reflect a real compartment fire scenario given the difference in the time-temperature curves between standard and real fires.

Current building code requirements for determining the fire resistance of structural systems are based on the reaction of specimens to a standard fire exposure such as defined by test standards ASTM E119, ISO 834, and NFPA 251. These standards have been the fundamental basis for determining fire resistance ratings (FRR) in building code applications since the 1920's. Although these standards have resulted in a reasonable level of safety given the lack of frequent building failures, there is nevertheless a growing body of evidence, which suggests that the entire testing procedure used by these standards is not realistic. Specifically, the time-temperature curves used by the standards do not compare well to the time-temperature curve of a real compartment fire. The result is that building construction may be needlessly costly. Some of the criticisms are:

- They are based on a specified time-temperature exposure that is not consistent with the characteristics of a real fire. Figure 1 illustrates the typical fire growth in a compartment, it can be seen that there is a significant decay phase after the burnout is complete. Standard fires specified in codes do not capture this (Boring *et al.*, 1981);
- Only single members are tested at a load corresponding to the maximum permissible stress of the member being tested, which is not representative since load bearing structural members in a building are not typically designed to carry a load at the maximum permissible stress, nor is the building load distributed evenly throughout the structural members (Pettersson and Wittenveen, 1979/1980; Steel Construction Institute, 2000);
- The length of time that a structural member can withstand the standard fire exposure while satisfying specific performance criteria defines the members fire resistance rating. This rating is a function of furnace construction and assumes that all furnaces are constructed equally (Drysdale, 1985).

It is also worth noting that ASTM E-119 states the following:

This standard should be used to measure and describe the response of materials, products, or assemblies to heat

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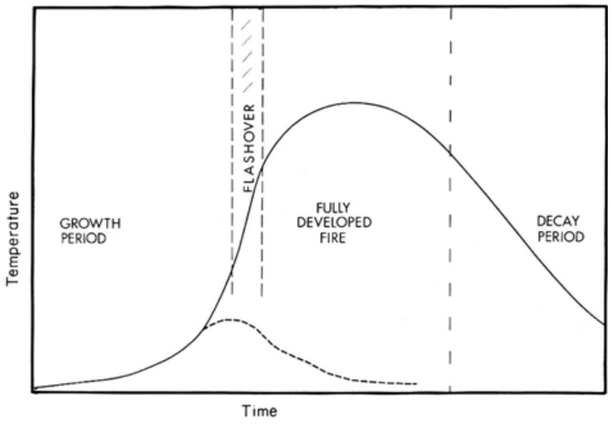

Figure 1. Typical Compartment Fire Time-temperature Curve (Thomas & Bennetts, 1991).

and flame under controlled conditions and should not be used to describe or appraise the fire-hazard or fire-risk of materials, products, or assemblies under actual fire conditions. However, results of the test may be used as elements of a fire-hazard assessment or a fire-risk assessment which takes into account all of the factors which are pertinent to an assessment of the fire hazard or fire risk of a particular end use.

2. Performance-based Design Philosophy

Typically, the performance requirements related to fire safety contained in performance-based standards are:

1. To ensure that a fire once started will not spread beyond the room of origin
2. To ensure that occupants will be given early warning of a fire occurrence to enable rapid evacuation from the building.
3. To ensure that the structure will remain standing long enough to allow occupants to escape;
4. To ensure that the structure will remain standing long enough for emergency personnel to perform their duties; and
5. To ensure the burning building will not fall down upon, or ignite the neighbouring properties.

Similarly, for structural safety the requirement is to reduce the probability of structural failure, and design the structure in a way that will ensure that the entire structural system will remain stable when a localized collapse occurs.

The current prescriptive building codes specify the required FRR for floor and wall assemblies, and structural members based on occupancy, building height, and building construction. Typically these start at a minimum 45 min FRR, followed by 1 hr, 1-1/2 hr, 2 hr and 4 hr ratings, and are applied throughout the building regardless of whether or not the rating is adequate based on actual fire load, risk etc.. To offer a justifiable alternative to this approach that will satisfy the objectives stated above, a performance-based design should be based on the following:

1. A fire scenario must be characterized by predicting fire load, fire size, fire severity and fire duration, and a time-temperature relationship for the fire scenario must be calculated;
2. The fire must be modeled in a location that represents a worst-case design for the building. That is, consideration must be given to both structural and fuel load to ensure the modeled compartment is representative of the building. To do this, multiple compartments should be assessed since the worst case fire location is not necessarily a structurally critical region in the building;
3. The time-temperature relationship of the fire exposed steel must be calculated and the thermal response determined relative to the known failure criteria of the member under consideration. Failure times must be based on a clearly defined "pass/fail criteria"; and
4. Use of the "inherent" or implied safety of the prescriptive code as the minimum level of safety to achieve. This can be done by utilizing the fire resistant ratings defined by the prescriptive code as a benchmark for the performance-based code.

3. Fire Scenario Development

In order to provide practical use for this design method, proper fire scenario development will be critical to achieving realistic results. To do this simplistic hand calculation procedures that can easily be set up in a spreadsheet format will be used instead of complicated computer programs.

3.1. Compartment fires

Typically a fire in a residential, commercial, or institutional building starts in a single compartment. This single compartment may be a bedroom in a home, an office in a commercial building, or classroom in an institutional building. Also needing consideration are corridors, which are long and narrow, and large lecture halls or conference rooms, which can be quite voluminous relative to a standard office or classroom. Although a window to the exterior may not always be present in one of these compartments, there is always a door, which may or may not be open at the time of the fire. The significance of the compartment geometry and number and location of openings has a direct impact on the behavior and severity of the fire.

In a typical compartment, with no openings the fire will burn more slowly and with less intensity and may self-extinguish as a result of the reduced oxygen supply in the room. In a long narrow room such as a corridor the fire tends to start burning available combustibles at the end of the compartment closest to the compartment opening, then migrating down to the opposite end of the corridor. This movement of flame and heat is drastically affected by the size of and location of openings (SFPE, 1995). Additionally, if a compartment is large enough relative to the fire size the fire will act as if in the open.

A serious fire in a compartment will typically have three distinct phases as follows:

1. Growth Phase: the fire is starting to grow from its point of origin and the temperature within the compartment is beginning to rise;
2. Fully Developed Phase: flashover has likely occurred and the compartment and all of its contents are engulfed in flame; and
3. Decay Phase: the period during which the compartment temperature starts to decrease as the fire consumes all available fuel and begins to loose energy.

3.2. Ventilation vs. Fuel Controlled Fires

The type of mathematical relationship that can be used to develop a time-temperature curve for the actual design fire is dependant upon whether the fire can be defined as ventilation or fuel controlled. A fire can be described as ventilation controlled when the burning rate is controlled by the available supply of oxygen necessary for combustion, and fuel controlled when the burning rate is controlled by the availability of fuel, under a fully ventilated condition.

Based upon experimentation; compartments with fuel loads ranging between 40 kg/m^2 to 100 kg/m^2 usually experience ventilation controlled fires. Furthermore, a ventilation-controlled fire is usually the most severe fire when analyzing a fire in a single compartment. This is the case because in a fuel-controlled fire the excess air entering the compartment is likely to have a cooling effect on the room temperature (Drysdale, 1985).

3.3. Room fuel load

One of the factors affecting the duration and intensity of the fire will be the room fuel load, which primarily consists of both fixed and moveable loads. The definition of each is described below:

• Fixed Fuel Load - consists of built-in combustible material such as floor and wall finishes, and permanently installed equipment such as lights, receptacles, ventilation diffusers, etc. Typically, this potential fuel is rarely moved or changed unless building renovations are undertaken.

• Moveable Fuel Load - this is the fuel load, which may vary during the life of the compartment under consideration as it generally consists of chairs, desks, books, wall hangings, etc.

To a lesser extent, the impact of both protected and unprotected materials may contribute to the fuel load. Protected fuel loads are combustible materials that are protected by some type of non-combustible cladding. The contribution of this load to the fire is a function of the probability that the protection will fail. Currently there is no accurate value that is available to describe this probability of failure (CIBW14 Workshop, 1986). Unprotected fuel loads are those loads that lack cladding or use combustible cladding. As with the definition for protected fire loads, the contribution of this load is a function of the probability that the protection will fail. A conservative estimate is to assume this type of cladding will always fail.

To calculate the design room fuel load per unit floor area within a compartment, the following equation can be based on the calculated masses of the various combustibles within the room using Table 1(Fitzgerald, 1999).

$$L_{fd}=\frac{1.58}{A_f}\sum M_i H_{ui} \tag{1}$$

where:

L_{fd} = calorific fuel load per unit floor area (MJ/m^2)
A_f = compartment floor area (m^2)
M_i = mass of product of combustion i (kg)
H_{ui} = lower calorific value of combustible material (MJ/kg)

Table 1. Estimating compartment fuel load (Fitzgerald, 1999)

Description	Cellulosic (kg)	Petro-Chemical (kg)
Building Fuels		
Structural Fuels	______	______
Service Fuels	______	______
• Non-Structural Fuels		
• Non-load bearing	______	______
Interior Finish & Trim	______	______
Contents Fuels		
Furnishings		
• Furniture	______	______
• Decorations	______	______
• Other	______	______
Occupant Related Goods		
Sub-total (kg)	______	______
	Conversion to Wood (kg)	______
Wood Equivalent Energy Content - (based on 18 MJ/kg)	______	______
Fuel Load (MJ)		

Note: (1) The mass of petro-chemical based materials is adjusted by a factor of 4.

Table 2. Summary of Variable Fuel Loads (per unit floor area)

	Variable Fuel Load (MJ/m^2)				
	Occupancy				
	Hospital-Patient Rm.	Hotel-Bedroom	General Office	Office-Average	Schools
New Zealand	-	-	-	475	-
Swiss	330	330	750	-	250
European	230	310	380-420	417	415
Swedish	-	310	417	411	555
USA	-	-	415	555	-

A report carried out by the Building Research Association of New Zealand (Narayanan, 1995) compared the fuel load survey from a small sample of New Zealand Life Insurance Offices to the CIB W14 study. Table 2 has combined the results of the New Zealand report with data from the CIB W14 study.

Although it is not recommended that these values be used explicitly in the development of compartment fire load as part of an engineering design, the values nevertheless provide a range that could be considered as typical for design purposes.

The New Zealand building code suggests the following (Buchanan, 2001):

- Residential Occupancy: 400 MJ/m^2 floor area
- Office Occupancy: 800 MJ/m^2 floor area
- Retail Occupancy: 1,200 MJ/m^2 floor area

Eurocodes recommend fuel load values range from 250 to 2,000 MJ/m^2 of compartment surface area. In some cases the values are also stated per unit floor area as indicated in Table 2.

4. Fully Developed Fire Modeling

The post-flashover, or fully developed fire, possesses the greatest risk to structural elements in a building due to the high temperatures generated during this stage of the fire. The compartment time-temperature model that reasonably characterizes this relationship for an actual compartment fire will provide the necessary engineering tool for this method. Typically, compartment time-temperature models have fundamental simplifying assumptions as follows (Pettersson *et al.*, 1976):

- combustion is complete and takes place exclusively inside the compartment;
- the compartment is well stirred so that the temperature is uniform throughout;
- the heat transfer coefficient of the compartment surfaces is a constant and uniform throughout the compartment; and
- the heat loss through the compartment boundaries is uniformly distributed.

In the early 1990's, draft Eurocodes addressing design issues related to structural steel for fire conditions were developed as follows:

- Eurocode 1: Basis of Design and Actions on Structures, Part 2.2: Actions on Structures Exposed to Fire; and
- Eurocode 3: Design of Steel Structures, Part 1.2 Structural Fire Design

Subsequently, the European Convention for Constructional Steel (ECCS) Model Code on Fire Engineering has been prepared to act as a follow-up to the Eurocodes. This document provides improvements to the approaches identified in the Eurocodes to reflect the improved understanding from research that has taken place since the introduction of the original Eurocodes. The time-temperature curve proposed is:

$$T_t = 20 + 1325(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}) \tag{2}$$

where

$$t^* = t \times \Gamma \tag{3}$$

$$\Gamma = \frac{(F_v/0.04)^2}{(b/1160)^2} \tag{4}$$

and

F_v = Ventilation factor, m$^{1/2}$
b = $k\rho c$ thermal inertia, J/m^{2}s$^{1/2}$K

the decay rates are:

$$T_t = T_{\max} - 625(t^* - t^*_{\max} \cdot x) \quad \text{for } t^*_{\max} \le 0.5 \tag{5}$$

$$T_t = T_{\max} - 250(3 - t^*_{\max})(t^* - t^*_{\max} \cdot x) \quad \text{for } 0.5 \le t^*_{\max} \le 2.0 \tag{6}$$

$$T_t = T_{\max} - 250(t^* - t^*_{\max} \cdot x) \quad \text{for } t^*_{\max} \ge 2.0 \tag{7}$$

where

$t^*_{\max} = 0.2 \times 10^{-3} \cdot (L_{t,d}/F_v) \cdot \Gamma$ and

$x = 1.0$ if $t_{\max} > t_{\lim}$, or $x = t_{\lim} \cdot \Gamma / t^*_{\max}$ if $t_{\max} = t_{\lim}$

where:
$t_{\lim}$ = 25 min for a slow growth fire
$t_{\lim}$ = 20 min for a medium growth fire
$t_{\lim}$ = 15 min for a fast growth fire

The model is applicable for the following conditions:

- Fire compartment floor areas are <500 m^2;
- Openings are only present in the vertical plane;

• Limited to fire compartments with mainly cellulosic type fire loads;
• Thermal inertia: $400 \le b \le 2000$ J/m^{2}s$^{1/2}$K;
• Opening factor: $0.02 \le F_v \le 0.2$; and
• The compartment boundaries are constructed of one material.

Some work has been done to calibrate the COMPF2 (Babrauskas, 1979) computer program to realistic compartment fires with respect to developing modifications to the Eurocode design fire curve described. In this work, which is described in detail by Buchanan and Feasy, a comparison of the existing Eurocode formulation as described in ENV 1991-2-2 with the output from the COMPF2 program was completed. This comparison identified a stark difference between the actual versus predicted fire time-temperature curves. As a result of these differences, there are two primary recommendations that have been proposed to address the discrepancy as follows:

$$\Gamma = \frac{(F_v/0.04)^2}{(b/1900)^2} \tag{8}$$

and that the decay phase of the fire indicted in equations be modified by the following:

$$\Gamma = \frac{\sqrt{(F_v/0.04)}}{\sqrt{(b/1900)}} \tag{9}$$

5. Critical Temperature of Fire-exposed Structural Steel

In the previous sections a method was demonstrated that can be used to predict a realistic, conservative time–temperature curve for a compartment fire based on the specific compartment dimensions, construction, fuel load and opening sizes. From this information, it is now necessary to derive the temperature history of the fire-exposed structural element based on the heat input resulting from the compartment fire.

During a fire, steel, whether in the form of a column, beam, or truss, will be exposed to hot gases from the fire. Given the high thermal conductivity of steel it is usually assumed that steel will be heated uniformly (Lie, 1992) resulting in a uniform temperature increase throughout the steel member. As a fire within a compartment intensifies, the mechanical properties such as tensile and yield strength, and modulus of elasticity, decrease. If the yield stress decreases to the working stress (about 50% of initial strength), the element will fail. The steel temperature at this moment is usually taken as the critical temperature. The critical temperature of steel is often taken as ~540°C, but varies depending upon the type and size of the steel member. This form of failure is known as the instantaneous deformation concept with limitations as follows:

Table 3. Critical temperatures for various types of steel

Steel	Standard/Reference	Temperature
Structural steel	ASTM	538°C
Reinforcing steel	ASTM	593°C
Pre-stressing steel	ASTM	426°C
Light-gauge steel	Eurocode 3	350°C

The model provides a general indication of when the failure in the structural member is likely to occur, but not the degree to which the member will deform during this failure process; and

The model does not provide insight into the condition of a structural member that is heated to just at or below the critical temperature maintained at this temperature and then cooled.

By maintaining the steel temperature below the critical temperature it is possible to ensure that the yield strength is not reduced to below 50% of the ambient value (Kodur, 2001). From a design perspective the critical temperature of steel varies depending upon the various types of steel as follows: (Pattersson *et al*., 1976)

6. Time-temperature History of Fire Exposed Members

There are numerous configurations under which structural steel may be found within standard building construction. Typical of these are:

Uninsulated steel structures, such as exposed columns, trusses, or beams; and

Insulated steel structures, such as columns, trusses, or beams with an applied fire protective layer.

6.1. Uninsulated Steel Members

The general heat balance equation has been given that equates the heat incidence upon the steel structure to the heat required to raise the steel temperature assuming uniform temperature distribution as follows:

$$\Delta T_S = \frac{F_s}{V_s}\frac{1}{\rho_s c_{ps}}\{h_c(T_t - T_s) + \sigma\varepsilon(T_t^4 - T_s^4)\}\Delta t\ (^{\circ}\mathrm{C}) \tag{10}$$

where:

F_s = Surface area of steel exposed to the fire per unit length, m^2/m
V_s = Volume per unit length of the steel section, m^3/m
h_c = Convective heat transfer coefficient, W/m^{2}K
T_t = Compartment temperature at time t, °C
ΔT_s = Steel temperature at time t, °C
Δt = Time step, hr
s = Steffan-Boltzman constant, kW/m^{2}K^4
e = Emissivity, --
ρ_s = Steel density, kg/m^3
c_{ps} = Specific heat of steel, J/kg°C

The above expression assumes that:

1. The steel temperature is uniformly distributed throughout the steel cross section; and

The convective heat transfer coefficient typically has a value between 20 W/m^{2}K to 25 /m^{2}K. The emissivity value is dependant upon both the flame and steel emissivities. A summary of acceptable values is contained in Table 4.

As steel is heated, its specific heat capacity changes while its density remains essentially unchanged at 7850 kg/m^3. To address this heating effect on specific heat capacity a temperature dependant calculation is proposed (Draft for Eurocode 3: part 1.2 - August 1993) for thermal capacity ($r_s c_{ps}$) as follows:

for $20 \le T_s < 600°C$

$$c_{ps} = 425 + (0.733T_s - 1.69 \times 10^{-3} T_s^2 + 2.22 \times 10^{-6} T_s^3) \text{ (J/kg K)} \quad (11)$$

for $600°C \le T_s < 735°C$

$$c_{ps} = 666 + 13002/(738 - T_s) \text{ (J/kg K)} \quad (12)$$

for $735°C \le T_s < 900°C$

$$c_{ps} = 545 + 17820/(T_s - 731) \text{ (J/kg K)} \quad (13)$$

for $900°C \le T_s \le 1200°C$

$$c_{ps} = 650 \text{ (J/kg K)} \quad (14)$$

Steel Section Ratio (F_s/V_s) represents a geometric ratio between the total surface area of the fire-exposed portions of the structural member and the volume per unit length. For full details on this calculation refer to "Fire Engineering Design of Steel Structures" (Pattersson *et al.*, 1976).

6.2. Insulated steel members

For insulated steel structures the methodology is similar to that of unprotected steel except that the impact of the insulating layer has to be accounted for in the final equation as shown below:

$$\Delta T_S = \frac{F_s}{V_s}\frac{k_i}{d_i \rho_S c_{ps}}\left\{\rho_s c_s \Big/ \left(\rho_s c_s + \frac{\frac{F}{V}(d_i \rho_i c_i)}{2}\right)\right\}(T_t - T_s)\Delta t \text{ (°C)} \quad (15)$$

The above expression assumes that:

1. The temperature gradient in the insulation is linear;
2. The temperature on the inside surface of the insulation is the same as the steel; and
3. The temperature on the outside surface of the fire protection equals the fire temperature.

The equation can be further modified by assuming that the energy stored in the insulation is small and can be neglected. Thus, the modified equation can be written as:

$$\Delta T_S = \frac{F_s}{V_s}\frac{k_i}{d_i \rho_S c_{ps}}(T_t - T_s)\Delta t \text{ (°C)} \quad (16)$$

where

k_i = Thermal conductivity of the insulation being assessed, W/mK

d_i = Thickness of insulating material, m

There are methods available to account for the potential storage of heat in insulating materials with higher heat capacity. However, it is more conservative to assume that all heat energy is transferred to the steel by ignoring this possibility.

The thermal conductivity of materials (k_i) typically used for the protection of structural steel are summarized in Table 5.

An incremental method will have to be used to solve these equations. The accuracy of the resulting answer will increase with smaller values for the time interval. Use of a spreadsheet will permit use of small time steps, typically 1/10th of the total fire duration and will yield acceptable results (Pattersson, 1978).

Using the time-temperature curve described previously, and following design criteria, the utility of the method is demonstrated for designing a steel column in an office building having the following features.

- Small three story office building having a FRR of 45 min. for structural/separating assemblies;
- Typical office with dimensions 5 m × 5 m × 2.75 m high having one fully exposed steel column in the room with a surface area to volume ratio of 50 m^{-1};
- Light compartment boundaries (gypsum wall board protected steel stud walls and lightweight concrete slab with open web steel joist and beam supporting structure)

Table 4. Resultant emissivity for fire exposed structural members (Pettersson *et al.*, 1976)

Type of construction	Resultant emissivity
Column exposed to fire on all sides	0.7
Column outside building façade	0.3
Floor girder with floor slab of concrete, only the underside of the bottom flange being directly exposed to fire	0.5
Floor girder with floor slab on the top flange	
Girder of I section for which the width-depth ratio is not less than 0.5	0.5
Girder of I section for which the width-depth ratio is greater than 0.5	0.7
Box girder and lattice girder	0.7

Table 5. Thermal conductivity of insulating materials

Material	Thermal conductivity (W/m°C)
Sprayed mineral fibre	0.1
Cementitious mixture	0.1
Perlite or vermiculite plates	0.15
Fibre silicate sheets	0.15
Wood	0.2
Gypsum wall board	0.2
Mineral wool slabs	0.25
Cellular concrete (600 kg/m^2)	0.30
Cellular concrete (1,000 kg/m^2)	0.45
Cellular concrete (1,300 kg/m^2)	0.65
Light weight concrete	0.80
Clay brick and lime brick	1.2
Normal weight concrete	1.3-1.7
Steel	35

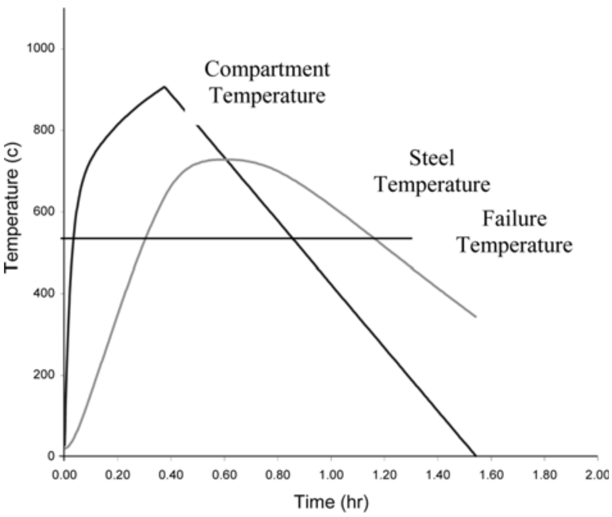

Figure 2. Comparison of time temperature curve for un-insulated steel exposed to Fire.

having a similar thermal inertia of 700 J/m^{2}s$^{1/2}$K;

• One opening consisting of a standard door at 2.13 m × 0.76 m; and

• A fuel load of 700 MJ/m^2 floor area based on wood equivalent value of 18 MJ/kg.

For the fire-exposed uninsulated steel case:

From this graph it is clear that the steel column will reach the critical temperature of 538°C at about 18 minutes, long before the required FRR is achieved, and as a result, protection is required.

Continuing with the example for uninsulated steel the required thickness of thermal insulation can be determined that will ensure the prescribed FRR is not exceeded. In this case, one 13 mm layer of gypsum wall board will provide a time to reach the critical temperature of 1 hour and 20 minutes as shown in the graph below.

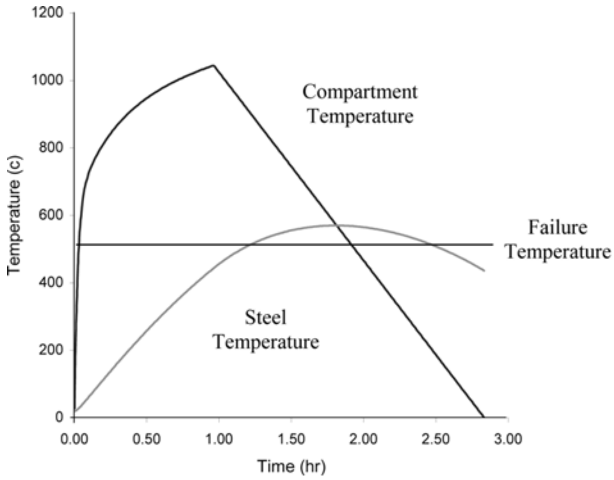

Figure 3. Comparison of time temperature curve for insulated steel exposed to fire.

7. Summary

The methods that have been summarized represent the first generation of design methods originating with work done by Kawagoe in the 1960's, and provide a single element analysis for fire protection purposes. There is, however, considerable effort currently under way to produce a second generation of design methods. For example the Steel Construction Institute in the UK has prepared a design manual specifically for multi-storey steel framed buildings made from composite construction. In this design manual the critical temperature of the composite assembly is coupled to the load ratio (actual strength at fire temperatures to load at ambient temperatures). This represents a level of refinement beyond that provided by this method.

As well, research is underway to more accurately account for the impact of end restraint conditions on the structural steel assembly, such as work done by Neves, who found that the critical temperature of steel columns can be influenced by the axial restraint and stiffness of the structure with reductions of ~20% for slender columns. Franssen concluded the same physical characteristics but determined that even though the column might fail earlier in the fire, (i.e. at a lower temperature) the assembly as a whole will not necessarily collapse due to load transfer from the column to the supporting structure. Others looking at rotational restraint of columns (Ali, 2001) have found that failure temperatures are higher under these conditions.

Notwithstanding the above, the method that has been described provides a simple approach that can be used by the design community to engineer fire protection requirements for fire-exposed structural components. Furthermore this method is a conservative approach grounded in over 20 years of empirical data. By coupling this method with the currently prescribed fire resistance ratings for structural elements contained in building codes, the designer should be able to bring the Authorities Having Jurisdiction on side with a certain level of comfort in the process.

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