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Capacity Reduction and Fire Load Factors for Design of Steel Members Exposed to Fire

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Abstract

A general reliability-based methodology is proposed for developing capacity reduction and fire load factors for design of steel members exposed to fire. The effect of active fire protection systems (e.g., sprinklers, smoke and heat detectors, fire brigade, etc.) in reducing the probability of occurrence of a severe fire is included. The design parameters that significantly affect the fire design of steel members are chosen as random variables. Raw experimental data published in the literature was analyzed to obtain the statistics of parameters for which no statistical information was available in the literature. Model errors associated with the thermal analysis models are also characterized based on experimental data. It is found that uncertainty associated with the fire design parameters is significantly higher than that of room temperature design parameters. To illustrate the proposed methodology, capacity reduction and fire load factors are developed for simply supported steel beams in U.S. office buildings, and it is shown that for consistent

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reliability these factors should vary depending on the presence of active fire protection systems in a building.

Keywords: Structural reliability, fire design, statistics of fire parameters, steel members

Introduction

The last decade has seen the promotion of performance-based codes for the fire design of steel members. These codes allow use of engineering approaches for fire design instead of prescriptive approaches (Ruddy et al. 2003) that are commonly used. For example, Appendix 4 of the 2005 AISC Specifications (referred to hereafter as "AISC Specifications") now allows steel members to be designed against fire using room temperature design specifications and reduced material properties. Similar provisions were developed by the European Convention for Constructional Steel work (ECCS 2001). Using this engineering approach, the verification of design for strength during fire requires that the load effects are less than the capacity of the structure. This leads to satisfying the design equation

$$W_{n,f} \leq \phi_f R_{n,f} \tag{1}$$

where $W_{n,f}$ is the load effect at the time of fire, $R_{n,f}$ is the nominal capacity at the time of fire, and ϕ_f is the capacity reduction factor. The AISC Specifications (AISC 2005a) allow using the same capacity reduction factors for fire design as those used for room temperature design. For example, ϕ_f = 0.9 is suggested for steel beams and columns. Most other codes suggest that a capacity reduction factor of 1.0 be used (e.g., in the Eurocode 3 (EN 2005), the partial safety factor γ_M is 1.0 for fire design). This recommendation is based on arguments that the probability of fire occurrence and the strength falling below the design value simultaneously is very small, and that fire design is based on the most likely expected strength (Buchanan 2001). Also, it is expected that live loads under fire conditions are likely to be smaller than those at room temperature conditions and hence there will be enough reserve strength available (Buchanan 2001). However, limited work has been done to develop capacity reduction factors based on reliability analysis (Magnusson and Pettersson 1981).

Fire safety is attained through two components: (1) active fire protection systems such as automatic sprinklers which help in controlling and suppressing the fire; and (2) passive fire protection systems such as structural and non-structural components of a building which control the spread of fire and prevent or delay the collapse of compartments. Passive fire protection can be achieved by protecting structural members in a variety of ways, e.g., by applying spray applied materials (sprayed mineral fiber, vermiculate plaster etc.), using intumescent coating, or using board materials (gypsum board) as insulation. The AISC Specifications suggest that while describing the design fire, due consideration should be given to the effectiveness of all active fire protection systems (sprinklers, smoke and heat detectors, etc.). The Commentary to Section 4.2.1.5 of Appendix 4 of the 2005 AISC Specifications states that while describing the design fire, the fire load may be reduced by up to 60 percent if a sprinkler system is installed in the building. Automatic sprinklers reduce the probability of occurrence of a severe fire. The reduction in fire load should be based on proper reliability analysis that includes the effect of sprinklers on the occurrence of a severe fire, and correspondingly on the probability of failure of structural steel members. Recently, a study was conducted in Europe through a research project of the European Coal and Steel Community (ECSC) (herein referred to as the ECSC study) to develop fire load factors by taking into account the variability of the fire load and the effect of active fire protection systems (ECSC 2001). However, the fire load factors were obtained using simplified assumptions and the study did not account for variability in other parameters. It is not apparent whether rigorous reliability analysis would yield results similar to those of the ECSC study.

A general methodology is presented in this paper for developing capacity reduction and fire load factors. In addition, the uncertainties of design parameters that significantly affect the fire design are characterized. The statistics of the random variables and model errors derived are then used for deriving capacity reduction and fire load factors for simply supported steel beams.

To better understand the performance functions, the engineering approach for designing steel members subjected to fire conditions is described next.

Engineering Approach for Designing Steel Members Exposed to Fire

In the engineering approach, the nominal capacity of steel members exposed to fire, $R_{n,f}$, is a function of fabrication parameters, F_i, and reduced material properties, $k_j(T_s)M_j$, and may be expressed as

$$R_{n,f} = f_R\left(F_1,\dots\dots F_l, k_1(T_s)M_1,\dots\dots k_k(T_s)M_k\right) \tag{2}$$

where the F_i are dimensional and sectional properties (e.g., depth of section, cross-sectional area, etc.), and M_j are the material properties at room temperature (e.g., yield strength, etc.). $k_j(T_s)$ are factors that account for reduction in strength and stiffness of steel at elevated temperature, and their values at different values of steel temperature, T_s, are given in the AISC Specifications.

According to the AISC Specifications, the design action (applied axial force, bending moment or shear force, etc.) is determined from the load combination given by

$$U = 1.2D + 0.5L + 0.2S + T \tag{3}$$

where, D, L and S are nominal dead, live and snow load effects, respectively, and T is the load effect induced by the fire itself (such as additional bending moment induced due to thermal expansions being restrained by the surrounding structure). The magnitude of the term T in Eq. (3) will depend both on the type of restraint and on the steel temperature, T_s. For of a simply supported beam, the term T in Eq. (3) will be zero because there will be no restraint effects under fire.

Under fire conditions, both the nominal capacity, $R_{n,f}$, estimated through Eq. (2) and the applied load effect, T, in Eq. (3), depend on the steel temperature, T_s, which in turn depends on the design fire (or time-fire temperature curve). The design fire depends on many factors such as ventilation conditions, thermal properties of the boundaries, the fire load (representative of combustible materials present), etc. As mentioned earlier, the Commentary to the AISC Specifications states that while describing the design fire, the fire load may be reduced by up to 60 percent if a sprinkler system is installed in the building. In a similar vein, Eurocode 1 (EN 2002) suggests a reduction in the fire load, while the ECSC study recommends either a reduction or increase in the fire load depending on the intended reliability. This reduction or increase (called fire load factor, γ_q, in this study) is to be applied to the fire load used in describing the design fire, and will affect the nominal capacity of all members and the fire-induced load effect, T, in Eq. (3) for restrained members.

At elevated temperatures, the strength and stiffness of steel reduces significantly, and if unprotected, steel members fail within a short time. Therefore, steel members are generally protected by fire protection material to slow down the rise of the steel temperature. The required thickness of the fire protection material can be determined using an iterative procedure, and the fire temperature in the compartment and the steel temperature of the member required for this procedure can be estimated as described in the next section.

Fire and steel temperatures

The fire temperature, T_f, can be estimated using a suitable mathematical model from the literature (SFPE 2000 and 2004). In this study, the Eurocode parametric fire model modified by Feasey and Buchanan (2002) is used to estimate the fire temperature under real fire scenarios. T_f is a function of the opening factor, F_v, fire load density, q_t, and thermal absorptivity, b.

The steel temperature can be calculated using any advanced finite element software. However, most design specifications such as the AISC Specifications and Eurocode 3 (EN 2005), allow the steel temperature to be calculated using simple thermal analysis methods such as the lumped heat capacity method.

The lumped heat capacity method assumes that the steel section is a lumped mass at uniform temperature. The heat balance differential equation for steel members protected by insulation can then be written as (Buchanan 2001)

$$\frac{dT}{dt} = \left(\frac{F}{V}\right)\left(\frac{k_i}{d_i \rho_s c_s}\right)\left\{\frac{\rho_s c_s}{\rho_s c_s + 0.5(F/V) d_i \rho_i c_i}\right\}(T_f - T_s) \tag{4}$$

where dT/dt = rate of change of steel temperature, F = surface area of unit length of the member (m^2) ,V = volume of steel per unit length of the member (m^3), ρ_s = density of steel (kg/m^3), c_s = specific heat of steel (J/kg.K), ρ_i = density of the insulation (kg/m^3), c_i = specific heat of insulation (J/kg.K), d_i = thickness of insulation (m), k_i = thermal conductivity of insulation (W/m.K), T_s = steel temperature (°C), and T_f = fire temperature (°C).

Eq. (4) can be written in finite difference form and the steel temperature can then be calculated at any time using a finite difference method that can be implemented in a spreadsheet. However, for incorporation into performance functions used in reliability analysis, a closed-form expression for calculating the maximum steel temperature is convenient. The closed-form solution of Eq. (4) was developed by Iqbal and Harichandran (2009).

Eq. (4) is used to estimate the temperature of steel members protected by insulation. The temperature of unprotected steel members can be estimated through a similar equation (Buchanan 2001). The heat balance differential equation for unprotected steel members is not presented since in the U.S. steel columns are always protected and steel beams are almost always protected.

The codes allow using the lumped mass method but caution that this method may be overly conservative for certain situations such as for a composite steel beam with a concrete slab on top in which a significant thermal gradient can occur through the depth. In this study, the lumped mass method is used because it is convenient within a reliability-based framework. The error arising from this method because of the assumption of a uniform temperature distribution is accounted for through a professional factor.

Methodology for Developing Capacity Reduction and Fire Load Factors

Development of capacity reduction and fire load factors involves three steps: (1) characterization of random design parameters; (2) selection of an appropriate performance function and characterization of the corresponding model errors; and (3) selection of a target reliability index or target probability of failure. These are described next.

Statistics of random parameters

The design parameters that significantly affect the fire design of steel members were chosen as random variables, and their means, coefficients of variation (COV), and distribution types are summarized in Table 1. The statistics of the arbitrary-point-in-time live load, fire load and ratio of floor area to total surface area of the fire compartment are specific to U.S. office buildings. The remaining parameters are general and apply to steel buildings of all use categories. The statistics of the dead and arbitrary-point-in-time live loads in Table 1 were reported by Ellingwood (2005) and Ravindra and Galambos (1978). We analyzed raw experimental data as discussed below to obtain the statistics of all parameters in Table 1 except for the fire load, arbitrary-point-in-time live load and dead load.

Fire Load

The fire load is based on the quantity of combustible materials present in a fire compartment, and is a measure of the total energy released in a fire. Culver (1976) reported statistics of the fire load for 23 typical U.S. office buildings. The fire load had a mean of 564 MJ/m^2 of floor area and a COV of 0.62. Culver reported the mean fire load in lb/ft^2 of floor area and we converted it to MJ/m^2 of floor area using a calorific value of wood of 17.5 MJ/kg. The ECSC study established that the fire load has a Gumbel distribution.

Ratio of Floor Area to Total Surface Area of the Compartment

Culver (1976) reported the fire load per unit floor area of the compartment. For calculating the fire temperature, the fire load needs to be converted to correspond to the unit area of the total surfaces of the compartment. This conversion can be done using

$$q_t = q_f \frac{A_f}{A_t} \tag{5}$$

where q_t = fire load per unit total surface area of the compartment, q_f = fire load per unit floor area of the compartment, A_f = floor area of the compartment, and A_t = total surface area of the compartment.

The ratio A_f/A_t varies for each compartment and should therefore be treated as a random variable in the reliability analysis. No statistical information is available in the literature about this ratio. Culver (1976) reported the range of floor areas for 23 office buildings in the U.S. but did not explicitly report the height of the rooms. Therefore, the height of the rooms was assumed to be 12 feet to establish the statistical parameters of the ratio A_f/A_t. Additionally, structural and architectural drawings of three representative office buildings in Detroit were examined to establish the mean, COV and distribution of the ratio A_f/A_t, and these were combined with those obtained from the data reported by Culver (1976). The combined mean, COV and distribution of the ratio A_f/A_t are given in Table 1.

Opening Factor

The opening factor, $F_v = A_v\sqrt{H_v} / A_t$, represents the ventilation conditions present in a fire compartment, where, A_v = area of the openings and H_v = height of the openings. The duration and severity of the fire depends on the value of the opening factor, which in turn depends on the sizes of windows and doors in a compartment. A building and its structural components are first designed for room temperature conditions and then for fire. The values of the opening factor for a fire compartment can be accurately estimated from the architectural drawings of a building and is not likely to be significantly different from the design or nominal values. Therefore, it is reasonable to treat the opening factor similar to the dead load in reliability analysis. For the opening factor we assumed the nominal values to be the mean values, a COV of 0.05, and a normal distribution.

Thermal Absorptivity of Compartment Enclosure

The thermal absorptivity, b, of the compartment boundaries is a measure of the amount of heat absorbed by the compartment boundaries and may be calculated through

$$b = \sqrt{k\rho c_p} \tag{6}$$

where, k, ρ and c_p are thermal conductivity, density and specific heat of the bounding material, respectively. The thermal absorptivity is a function of temperature, but the Eurocode 1 (EN 2002) allows room temperature properties to be used for design. We performed a detailed analysis to study the effect of the temperature variation of b on the steel temperature, and the results indicated that it is reasonable to use room temperature values of b.

There is no information available in the literature about the variability of b for different bounding materials. However, some researchers have reported thermal properties of some commonly used bounding materials such as normal and lightweight concretes and gypsum board. These reported room temperature thermal properties were used to characterize b for normal and lightweight concretes and gypsum board as described below.

Thermal properties (density, thermal conductivity and specific heat) of gypsum boards reported by different researchers (Carino et al. 2005; Manzello et al. 2008; Mehaffey et al. 1994; Thomas 2002; Wullschleger and Wakili 2008) were used to obtain the thermal absorptivity, b_g, through Eq. (6). The statistics of b_g based on these calculated values are shown in Table 1. The mean value of 423 $Ws^{0.5}/m^2K$ is close to the value of 410 $Ws^{0.5}/m^2K$ reported by Buchanan (2001) for gypsum board.

In case of normal and lightweight concretes, all three corresponding thermal properties were not available for a particular tested specimen. Therefore, first the statistics of density (ρ), thermal conductivity (k) and specific heat (c_p) for both types of concretes were obtained using test data. Thermal properties were reported by Harmathy and Allen (1973), Lie and Kodur (1996), Shin et al. (2002), Schneider et al. (1981), and Whiting et al. (1978) for normal weight concrete, and Harmathy and Allen (1973), Stukes et al. (1986), and Whiting et al. (1978) for lightweight concrete. For normal weight concrete, Schneider et al. (1981) included test data obtained in six other studies which was also used for characterizing thermal properties of normal weight concrete. The statistics of b for normal and lightweight concretes shown in Table 1 were obtained using the distributions of the density, thermal conductivity and specific heat through Monte Carlo simulations. The mean value of b_{NWC} = 1830 $Ws^{0.5}/m^2K$ compares well with the value of 1900 $Ws^{0.5}/m^2K$ reported by Buchanan (2001). The mean value of b_{LWC} = 640 $Ws^{0.5}/m^2K$ compares well with the value of 660.6 $Ws^{0.5}/m^2K$ reported by Kirby et al. (1994) for lightweight concrete blocks.

Buchanan (2001) studied the effect of two types of bounding materials (normal weight concrete having b = 1900 $Ws^{0.5}/m^2K$ and gypsum board having b = 410 $Ws^{0.5}/m^2K$) on the fire temperature. A typical commercial office building constructed from a mixture of these materials on the walls and ceiling would give values of fire temperature in between those obtained by using either of the individual materials (Buchanan 2001). Therefore, statistics of b were also obtained for a compartment assuming that 50% of the total surface area was constructed of normal weight concrete and the other 50% of gypsum board. The total thermal absorptivity of this compartment can be expressed as

$$b_{g+NWC} = \frac{0.5A_t b_g + 0.5A_t b_{NWC}}{A_t} \tag{7}$$

The mean and COV of b_{g+NWC} for this mixed compartment are also given in Table 1.

To study the fire and steel temperatures likely to occur in real fire scenarios, Kirby et al. (1994) conducted 9 fire tests using different materials (such as lightweight concrete blocks, autoclaved aerated concrete slabs, fluid sand, ceramic fiber, and fireline plasterboard) as walls, roof and floor of the compartment. The combined value of thermal absorptivity in all tests ranged from 350-755 $Ws^{0.5}/m^2K$. The statistics of b developed in this study effectively cover the values of b used by Kirby et al. (1994).

Thickness of Insulation or Fire Protection Materials

Steel members may be protected using either spray applied fire protection materials or board systems. Carino et al. (2005) studied the variation of thickness of spray applied fire protection materials used in the World Trade Center (WTC). They observed that the average thickness is generally higher than the specified thickness and that the thickness is distributed lognormally. Their results were used for the COV and distribution type for insulation thickness. Because the thicknesses of fire protection materials used in the WTC were determined using prescriptive approaches, the mean of the insulation thickness was not taken from this study. Instead, based on the analysis in the report and conversation with a fire protection expert (Ferguson 2008), the mean was taken to be 1/16-inch higher than the thickness required using performance-based design. There is no information available on the variability of thickness of board materials, but since they are produced under controlled conditions, the nominal thickness was taken as the mean value and the COV was assumed to be 0.05.

Thermal Conductivity and Density of Fire Protection Materials

Bruls et al. (1988) studied the variation of thermal conductivity at different temperatures. Although, thermal conductivity varies with temperature, they concluded that since the failure of structural steel elements generally occurs at a temperature of 400 to 600°C, the thermal conductivity corresponding to a critical temperature of 500°C can be used in design.

Statistical analysis of the thermal conductivity in the temperature range of 400-600°C for eight representative materials used in the U.S. (five reported by Bentz and Prasad 2007, two reported by Carino et al. 2005, and one tested at Michigan State University) was performed. The mean, COV and distribution type established from this statistical analysis are given in Table 1.

Room temperature values of density reported in the literature for different fire protection materials (Bentz and Prasad 2007, and Carino et al. 2005) were used to obtain its statistics. The mean and COV of the density of spray applied fire protection materials are given in Table 1. Due to insufficient data, it was not possible to estimate a distribution and a normal distribution was assumed.

Different types of board materials can be used as fire protection materials (e.g., fiber-silicate or fiber calcium silicate boards, and gypsum plaster (Buchanan 2001)). Thermal properties of all of these boards are generally not easily available because of their proprietary nature. However, thermal properties of gypsum boards have been reported by various researchers and were used to obtain the statistics of thermal conductivity and density. Statistics of thermal conductivity of gypsum board materials in the temperature range of 400-600°C were obtained using test data (Bentz and Prasad 2007, Carino et al. 2005, Manzello et al. 2003, Mehaffey et al. 1994, Sultan 1996, and Thomas 2002) and are shown in Table 1.

Room temperature values of the density of gypsum board reported by different researchers (Carino et al. 2005, Mehaffey et al. 1994, Thomas 2002, Tsantaridis et al. 1999, and Wullschleger and Wakili 2008) were used to obtain the statistics that are shown in Table 1.

Buchanan (2001) reported typical values of thermal conductivity to be 0.15 W/m.K and 0.20 W/m.K, and typical values of densities to be 600 kg/m^3 and 800 kg/m^3, for fiber-silicate or fiber calcium silicate boards, and gypsum plaster, respectively. The mean density of 745 kg/m^3 and mean thermal conductivity of 0.16 W/m.K fall within the range of reported values. Therefore, although the statistics of density and thermal conductivity were obtained using test data of gypsum boards only, they should adequately represent other types of board materials as well.

Performance function for reliability analysis

Applied Loads

Ellingwood (2005) showed that the probability of coincidence of a fire with maximum values of live load, roof live load, snow, wind, or earthquake loads is negligible, and a structure is likely to be loaded to only a fraction of the design load when a fire occurs. Therefore, it is appropriate to use the combination of dead and arbitrary-point-in-time live load for reliability analysis under fire conditions. This is consistent with Beck's (1985) recommendation. Therefore, the load effect W_f for reliability analysis may be calculated as

$$W_f = E(c_D AD + c_L BL_{apt}) \tag{8}$$

where c_D and c_L = deterministic influence coefficients that transform the load intensities to load effects (e.g., moment, shear, and axial force), A and B = random variables reflecting the uncertainties in the transformation of loads into load effects, E = a random variable representing the uncertainties in structural analysis, and D and L_{apt} = random variables representing dead and arbitrary-point-in-time live load. The statistics of D and L_{apt} are given in Table 1. The statistics of parameters A, B and E are: (1) mean of A = 1.0, COV of A = 0.04; (2) mean of B = 1.0, COV of B = 0.20; and (3) mean of E = 1.0, COV of E = 0.05 (Ravindra and Galambos 1978).

Capacity of Steel members

The actual capacity of steel members under fire can be obtained by modifying the nominal capacity given by Eq. (2) to

$$R_f = P.f_R\left(f_1F_1,\ldots\ldots f_lF_l, k_1(t_sT_s)m_1M_1,\ldots\ldots k_k(t_sT_s)m_kM_k\right) \tag{9}$$

where $P, f_i, m_j,$ and t_s are the non-dimensional random variables defined below.

P = "Professional" factor, reflecting uncertainties of the assumptions used in determining the capacity from design equations. These uncertainties may result from using approximations in place of exact theoretical formulas, and from assumptions such as perfect elasto-plastic behavior and a uniform temperature across the section.

f_i = Random variable that characterizes the uncertainties in "fabrication."

m_j = Random variable that characterizes uncertainties in "material properties."

t_s = Random correction factor that accounts for differences between the steel temperature obtained from models and that measured in actual tests.

Limit State Equation

Using Eqs. (8) and (9), the limit state equation for reliability analysis under fire conditions may be written as

$$g(\mathbf{x}) = R_f - W_f \tag{10}$$

where **x** denotes a vector containing all the random variables. The probability of failure, p_f, of a steel element under fire is $p_f = P[g(\mathbf{x}) < 0]$.

It is assumed that the random variables f_i and m_j are the same as those used for developing LRFD specifications for ambient temperature conditions and their statistics are available in the literature. The statistics of P are specific to each design equation, cannot be generalized, and can be obtained from a comparison between the predicted capacity and test results. The statistics of t_s are characterized below.

Model Error for Steel and Fire Temperatures

The maximum temperature of steel sections estimated using Eq. (4) differs from that measured in actual fire tests due to: (1) the approximation and assumptions used in the models for estimating fire and steel temperatures; and (2) differences between predicted and actual heat absorbed, ventilation conditions, and duration of burning. To account for the differences in calculated and measured steel temperatures, the model error was characterized as described below, both for steel beams (three sided exposures) and steel columns (four sided exposure).

The experimental temperature of steel elements has been reported by many researchers but most tests were carried out under standard fires instead of real fires, and thus cannot be used to estimate the error arising from the fire models. Kirby et al. (1994) carried out a series of nine real fire tests and recorded the temperature of protected and unprotected steel elements. The tests were performed for a range of fire loads (380 – 760 MJ/m^2 of floor area), for different opening conditions (F_v = 0.0029 – 0.062 m$^{1/2}$), and various types of materials were used as compartment boundaries in order to represent all possible real fire scenarios. Foster et al. (2006) reported the temperature of four protected steel columns. In this test, the fire load was 720 MJ/m^2 of floor area, and the opening factor was 0.043 m$^{1/2}$.

The model error for the temperature of steel beams, t_{sb}, was characterized using the test data reported by Kirby et al. (1994), and the model error for the temperature of steel columns, t_{sc}, was characterized using the test data reported by Kirby et al. (1994) and Foster et al. (2006). t_{sb} has a mean of 0.98 and COV of 0.11, and t_{sc} has a mean of 1.05 and COV of 0.13. Both, t_{sb} and t_{sc} were best described by the Gumbel distribution.

In the last decade, many real fire tests were carried out all over the world, especially in the U.K. In most of these tests the steel beams were unprotected, and therefore, reported steel temperatures cannot be used for characterizing the model error for protected beams. In almost all tests, steel columns were protected but various parameters (e.g., type, thickness and properties of insulation, type, size and thermal properties of bounding materials) required as input data for estimating the temperature of columns were not explicitly reported. Therefore, the experimental temperatures recorded in these tests could not be used.

Probability of failure and target reliability index

The reliability index, β, is a relative measure of safety of a designed structural component, and is related to the probability of failure. On the other hand, the target reliability index, β_t, controls the safety factors used in design equations. CIB W 14 (1986) suggests that the rare occurrence of a severe fire should be taken into account while developing safety factors for fire design. The presence of active fire protection systems such as automatic sprinklers, fire brigade, etc., reduce the probability of occurrence of a severe fire and hence reduce the probability of failure. Therefore, the reduced probability of failure under fire can be accounted for by using a reduced target reliability index.

A detailed methodology for calculating the target reliability index, β_t, by incorporating the effect of active fire protection systems in reducing the probability of occurrence of a severe fire was presented in the ECSC study (ECSC 2001). The ECSC study also suggested appropriate values for the effectiveness of different active fire protection systems in reducing the probability of occurrence of a severe fire. Using the methodology described in the ECSC study, the target reliability indices were estimated for typical fire compartments (ranging in floor areas from 25-500 m^2) of U.S. office buildings. It was found that it is reasonable to use target reliability index values ranging from zero to 2.0 for developing capacity reduction and fire load factors. Since the probability of occurrence of a severe fire varies depending on the presence of active fire protection systems, the target reliability index also varies for different design situations.

To account for the reduced probability of occurrence of a severe fire, a similar approach to the one presented in the ECSC study was suggested by Ellingwood and Corotis (1991) for fire resistant structural design. They suggested a probability of failure for fire situations that corresponds to β_t of about 1.5, which falls within the range of zero to 2.0 found from the ECSC study.

Capacity Reduction and Fire Load Factors for Simply Supported Beams

Predictions of structural capacity under fire are still relatively new and evolving. With improved understanding of structural behavior under fire, performance equations may change and future design refinements may be necessary. In this section, the bending capacity of simply supported beams given in AISC Specifications is used.

Performance function and statistics of random parameters

The nominal moment capacity of a simply supported, laterally restrained steel beam exposed to fire can be expressed as

$$M_{n,f} = Z_x k_y (T_s) F_y \tag{11}$$

where Z_x = plastic section modulus, F_y = yield strength of steel at room temperature, and $k_y(T_s)$ = yield strength reduction factor that depends on the temperature, T_s, of the steel member.

The actual moment capacity can be obtained by modifying Eq. (11) to

$$M_f = P f_z Z_x k_y (t_{sb} T_s) m_{F_y} F_y \tag{12}$$

where f_z is a random variable with a mean of 1.03 and COV of 0.034 that characterizes uncertainty in Z_x (Schmidt and Bartlett 2002), m_{Fy} is a random variable with a mean of 1.03 and COV of 0.063 that characterizes uncertainty in F_y (Schmidt and Bartlett 2002) and t_{sb} is the model error for steel temperature with the statistics given earlier. Steel temperature, T_s, is a function of many parameters (see Eq. (4)) whose statistics are given in Table 1. P is the professional factor (model error) and is characterized in the next subsection.

The performance function can be written as

$$g(\mathbf{x}) = M_f - M_{a,f} \tag{13}$$

where $M_{a,f}$ is the applied moment under fire that can be expressed in terms of basic random variables as shown in Eq. (8).

Professional factor for moment capacity equation

To account for the difference in the measured capacity of a laterally restrained beam in a laboratory and that predicted by Eq. (11), the professional factor, P, representing their ratio was characterized using the test results reported by Kruppa (1979) and Wainman (1992). Kruppa (1979) reported test results for sixteen beams and Wainman (1992) reported the test results for two beams. P has a mean of 0.99 and a COV of 0.11, and is best described by the lognormal distribution.

Reliability analyses

Ten laterally restrained beams ranging in length from 3 m (10 ft) to 13.7 m (45 ft) and live loads ranging from 2.4 kPa (50 psf) to 4.8 kPa (100 psf) were selected for the reliability study. The AISC Specifications were used to first design the beams for ambient temperature conditions. The same beams were then designed for fire exposure (b = 640 W/m.K and F_v = 0.02 m$^{1/2}$) and the required thickness of insulation to withstand the design fire was determined using the procedure described in Appendix 4 of the AISC Specifications (AISC 2005a) (i.e., the engineering approach described earlier). The beams were assumed to be protected by spray applied fire protection materials, which is generally the case in the U.S.

The FERUM (Finite Element Reliability Using Matlab) software (Der Kiureghian 2006) was used to perform the reliability analysis. FERUM is a general purpose structural reliability software written using Matlab. It can be used to perform reliability analysis using different methods, including the first-order reliability method (FORM).

The detailed framework for first-order reliability analysis and the simplified expressions for obtaining the partial safety factors for each design parameter are described in NBS 577 (Ellingwood et al. 1980), and is not reproduced here. FORM analysis was performed for each design situation (each of the 10 beams) using FERUM. The partial safety factors for each design parameter were obtained using the methodology described in NBS 577 for each design situation. These individual partial safety factors, except for the fire load, were then combined into a single capacity reduction factor. Thus 10 different capacity reduction factors (one for each beam) were obtained. Thereafter, a single optimized capacity reduction factor corresponding to dead and live load factors of 1.2 and 0.5, respectively, was obtained using the optimization procedure described in NBS 577 for each β_t value ranging from zero to 2.0. A similar procedure was used to obtain the fire load factors corresponding to each β_t value.

Fire load is a major parameter in fire design, and uncertainty associated with the fire load has a significant effect on the safety of the design. Therefore, the variability of the fire load on overall safety is accounted for through the specific partial safety factor on the fire load. As mentioned in the introduction, the Commentary to the AISC Specifications, Eurocode 1, and the ECSC study, the fire load may be reduced to account for the effect of active fire protection systems installed in the building. These recommendations also motivated use of a separate safety factor for fire load. The fire load factor, γ_q, is to be applied to the fire load used in describing the design fire and will affect the nominal capacity.

Results

Capacity reduction factor

The plot of the capacity reduction factor, ϕ_f, vs. the target reliability index, β_t, is shown in Fig. 1 and is given by

$$\phi_f = \begin{cases} 1.0 & \text{for } \beta_t \le 1.25 \\ 1.5 - 0.4\beta_t & \text{for } 1.25 \le \beta_t \le 2.0 \end{cases} \tag{14}$$

Most codes suggest that ϕ_f = 1.0 be used. However, ϕ_f = 0.9 is suggested in the Commentary to the AISC Specifications. Results obtained in this study indicate that the nominal capacity need not be reduced (i.e., ϕ_f = 1.0) if β_t is less than 1.25, which in turns depends on the effectiveness of active fire protection systems in reducing the probability of occurrence of a severe fire.

Using different capacity reduction factors for different design situations may not be desirable from a codification point of view. We described earlier why β_t should be varied to account for the presence and effectiveness of active fire protection systems. The purpose of developing the capacity reduction factor for a range of β_t values (0 to 2) instead of a single value was to provide options to specification writers. The code authorities may decide to use one β_t value (e.g., a value of about 1.5 as suggested by Ellingwood and Corotis (1991)) depending on their comfort about the effectiveness of active fire protection systems, and thus specify a constant capacity reduction factor for each limit state. Since most U.S. buildings are equipped with reliable sprinklers, β_t is not likely to exceed 1.5 and it may be appropriate to use the capacity reduction factor corresponding to β_t = 1.5.

Fire load factor

The plot of the fire load factor, γ_q, vs. the target reliability index is shown in Fig. 1. The nominal value of the fire load was taken as the 90th percentile (Buchanan 2001). The value of γ_q for a given β_t is

$$\gamma_q = \begin{cases} 0.4 + 0.4\beta_t & \text{for} \quad \beta_t \le 1.25 \\ 0.15 + 0.6\beta_t & \text{for} \quad 1.25 \le \beta_t \le 2.0 \end{cases} \tag{15}$$

When the target reliability index is less than 1.42, the fire load factor given by Eq. (15) is less than 1.0 indicating that the fire load can be reduced as suggested in the ECSC study and Eurocode 1. The commentary to the AISC Specifications states that the fire load may be reduced by up to 60% if a sprinkler system is installed. The maximum reduction should be considered only when the automatic sprinkler system is considered to be of the highest reliability, i.e., having reliable and adequate water supply, supervision of control valves, and regular schedule for maintenance in accordance with NFPA recommendations (NFPA 2002). The reduction in fire load specified in Fig. 1 depends on the target reliability index, which in turn depends on the effectiveness of active fire protection systems in reducing the probability of occurrence of a severe fire. The proposed approach is more general and enables the reduction in fire load to be specified for sprinkler systems of all categories, i.e., having low, high or medium reliability, as well as for other active fire protection systems.

Validity of capacity reduction and fire load factors for multiple fire scenarios

To account for different bounding surfaces and ventilation conditions, we used three values of b (423 $Ws^{0.5}/m^2K$, 640 $Ws^{0.5}/m^2K$ and 1160 $Ws^{0.5}/m^2K$) and three values of opening factors (0.04 $m^{1/2}$, 0.08 $m^{1/2}$ and 0.12 $m^{1/2}$) to obtain nine fire scenarios which were then used to validate the capacity reduction and fire load factors derived above. For these nine fire scenarios, two beams were designed for fire conditions using the capacity reduction and fire load factors shown in Fig. 1. Thus, for each β_t value, we had 18 design situations, and a total of 90 design situations for five β_t values. Reliability analysis was then performed and the computed reliability index values, β, for both beams are compared with the β_t values in Fig. 2.

The β values compare quite well with the β_t values, indicating that the derived capacity reduction and fire load factors work for all design situations considered. The β values are conservative for β_t values less than about 1.5. For β_t values of less than 1.5 (see Fig. 1), the ϕ_f found from reliability analysis was greater than 1.0, and the nominal capacity could be increased. However, since ϕ_f is generally always taken to be less than or equal to 1.0 in LRFD specifications, we restrained the ϕ_f for fire design to also not exceed 1.0. Because of this inherent conservatism, the β values are higher than the β_t values.

Comparison of Fire Load Factors with those Based on ECSC Method

The fire load factor in the ECSC study was obtained using simplified assumptions instead of rigorous reliability calculations, and was specified for any β_t value through

$$\gamma_q = 1.05 \frac{\left\{1 - \frac{\sqrt{6}}{\pi} V_q \left[0.577 + \ln(-\ln \Phi(0.9\beta_t))\right]\right\}}{\left\{1 - \frac{\sqrt{6}}{\pi} V_q \left[0.577 + \ln(-\ln(p))\right]\right\}} \tag{16}$$

where V_q = COV of the fire load, Φ = cumulative standard normal distribution function, and p = percentile used for obtaining the characteristic or nominal fire load. If the nominal value is taken as the 90th percentile, then $p = 0.9$.

In Fig. 3, the γ_q obtained in this study is compared with that obtained using Eq. (16) for U.S. fire load statistics taking the nominal value of fire load to be the 90th percentile. γ_q obtained from the ECSC method is greater than that derived in the this study for β_t values smaller than about 1.5, and is almost the same for β_t values greater than 1.5. As shown earlier, γ_q derived in this study yields conservative β values for β_t values less than about 1.5, and hence the γ_q obtained according to the ECSC approach will yield even more conservative results. For β_t values greater than 1.5, the beams designed as proposed herein yield the intended safety level or higher (see Fig. 2) because γ_q is used in combination with a ϕ_f shown in Fig. 1 that is less than 1.0.

Reliability Inherent in AISC Fire Design Methodology

It is of interest to determine what β value is inherent in the AISC approach. The insulation thicknesses for six beams for the 9 different fire scenarios described earlier were determined using the AISC approach for two cases: (1) using $\phi_f = 0.9$ and by reducing the 90th percentile of the fire load by 60% (for sprinklers of the highest reliability) as suggested in the Commentary to the AISC Specifications, and (2) using $\phi_f = 0.9$ and using the 90th percentile of the fire load assuming that there are no reliable sprinklers in the building. Reliability analysis was then performed using these insulation thicknesses, and it was found that the reliability index varied from 0.2 to 0.5 for Case 1 and from 1.45 to 1.60 for Case 2.

Summary and Conclusions

A general reliability-based methodology is proposed for developing capacity reduction and fire load factors for the design of steel members exposed to fire. Statistics of a variety of parameters important for the design of steel members under fire were obtained from experimental data reported in the literature. Model errors associated with the thermal models were also characterized based on experimental data. It was found that uncertainty associated with the fire design parameters is much higher than that associated with room temperature design parameters. The capacity reduction and fire load factors correspond to a preselected target reliability index that accounts for the effect of active fire protection systems (e.g., sprinklers, smoke and heat detectors, etc.) in reducing the probability of occurrence of a severe fire.

To illustrate the proposed methodology, capacity reduction and fire load factors are derived for simply supported steel beams in U.S. office buildings exposed to fire. It is found that the fire load factor should vary depending on the presence of active fire protection systems. This is in agreement with the Commentary to the AISC Specifications, the Eurocode 1, and the ECSC study. It is found that the capacity reduction factor should also vary when active fire protection systems are present.

For most office building compartments in the U.S. equipped with sprinklers, use of $\phi_f = 1.0$ is reasonable, and γ_q is likely to lie between 0.4 and 1.0.

Current structural fire design provisions are still relatively new and evolving, with various remaining uncertainties and information gaps. The methodology proposed herein is an initial attempt to characterize uncertainties in current fire design provisions. It is expected that as the research in this field yields improved understanding of structural behavior under fire, future design refinements will be necessary.

References

AISC (2005a). "Load and resistance factor design specifications for structural steel buildings." American Institute of Steel Construction, Inc.

AISC (2005b). "Commentary on the load and resistance factor design specifications for structural steel buildings." American Institute of Steel Construction, Inc.

Beck, V. R. (1985). "The prediction of probability of failure of structural steel elements under fire conditions." *Transactions of the Institution of Engineers,* Australia, 27 (1), 111-118.

Bentz, D. P., and Prasad, K. R. (2007). “Thermal performance of fire resistive materials I. Characterization with respect to thermal performance models.” *Report No.: NISTIR 7401*, National Institute of Standards and Technology.

Bruls, A., Cajot , L. G., and Franssen, J. M. (1988). “Characterization of an insulating material with regard to ECCS recommendations for the fire safety of steel structures.” *Journal of Constructional Steel Research,* 9(2), 111-135.

Buchanan, A. H. (2001). *Structural design for fire safety*, John Wiley & Sons Ltd., New York.

Carino, N. J., Starnes, M. A., Gross, J. L., Yang, J. C., Kukuck, S. R., Prasad, K. R., and Bukowski, R. W. (2005). “Federal building and fire safety investigation of the World Trade Center disaster: Passive fire protection.” *Report No.: NIST NCSTAR 1-6A* , National Institute of Standards and Technology.

CIB W14. (1986). “Design guide for structural fire safety.” *Fire Safety Journal*, 10(2), 81-137.

Culver, C. G. (1976). “Survey results for fire loads and live loads in office buildings.” National Bureau of Standards.

Der Kiureghian, A., Haukaas, T., and Fujimura, K. (2006). “Structural reliability software at the University of California, Berkeley.” *Structural Safety*, 28(1-2), 44-67.

ECCS Technical Committee 3 (2001). “Model code on fire engineering, 1st edition.” Document No. 111, European Convention for Constructional Steelwork, Brussels, Belgium.

ECSC. (2001). “Natural fire safety concept.” Valorization research project. Product-Structural Dept. ARBED-Research Centre L-4009 ESCH/ALZETTE.

Ellingwood, B. R. (2005). “Load combination requirements for fire-resistant structural design.” *Journal of Fire Protection Engineering*, 15(1), 43-61.

Ellingwood, B. R. and Corotis, R.B. (1991). "Load combinations for buildings exposed to fires."

Engineering Journal, AISC 28(1):37-44.

Ellingwood, B. Galambos, T. V., MacGregor, J. G., and Cornell, C. A. (1980). "NBS special publication 577: Development of a probability based load criterion for American National Standard A58." National Bureau of Standards, Washington, DC.

EN (2002). "Eurocode 1: Actions on structures- Part 1-2: General actions- Actions on structures exposed to fire." BS EN 1991-1-2: 2002.

EN (2005). "Eurocode 3: Design of steel structures- Part 1-2: General rules-Structural fire design." BS EN 1993-1-2: 2005.

Ferguson, D. (2008). Verbal communication. Reicherbach Corporation, Lansing, Michigan, U.S.

Feasey, R. and Buchanan, A. (2002). "Post-flashover fires for structural design." *Fire Safety Journal*, 37(1), 83-105.

Foster, S., Chladna, M., Hsieh, C., Burgess, I., and Plank, R. (2007). "Thermal and structural behaviour of a full-scale composite building subject to a severe compartment fire." *Fire Safety Journal*, 42(3), 183-199.

Harmathy, T. Z., and Allen, L. W. (1973). "Thermal properties of selected masonry unit concretes." *ACI Journal*, 70(2), 132-142.

Iqbal, S. and Harichandran, R. S. (2009). "Reliability-based design specifications for simply supported steel beams exposed to fire." *Proceedings of 10th International Conference on Structural Safety and Reliability*, Osaka, Japan.

Kirby, B. R., Wainman, D. E., Tomlinson, L. N., Kay, T. R., and Peacock, B. N. (1994). "Natural fires in large scale compartments—A British steel technical, fire research station collaborative project." British Steel Technical.

Kruppa, J. (1979). "Collapse temperature of steel structures." *Journal of the Structural*

*Division,*105(ST9), 1769-1788.

Li, T. T., and Kodur, V. K. R. (1996). “Thermal and mechanical properties of steel-fibre-reinforced concrete at elevated temperatures.” *Canadian Journal of Civil Engineering*, 23(2), 511-517.

Magnusson, S. E., and Pettersson, O. (1981). “Rational design methodology for fire exposed load bearing structures.” *Fire Safety Journal*, 3(4), 227-241.

Manzello, S. L., Park, S-H., Mizukami, T., and Bentz, D. P. (2008). “Measurement of thermal properties of gypsum board at elevated temperatures.” *Proceedings, 5th International Conference on Structures in Fire*, Singapore, 656-665.

Mehaffey, J. R., Cuerrier, P., and Carisse, G. (1994). “A model for predicting heat transfer through gypsum-board/wood-stud walls exposed to fire.” *Fire and Materials*, 18(5), 297-305.

NFPA (2002). “Handbook of fire protection engineering, 3rd edition.” National Fire Protection Association, Quincy, MA.

Ravindra, M. K. and Galambos, T. V. (1978). “Load and resistance factor design for steel.” *Journal of the Structural Division,* 104(ST9), 1337-1353.

Ruddy, J. L., Marlo, J. P., Ioannides, S. A. and Alfawakiri, F. (2003). “Fire resistance of structural steel framing. Steel Guide No. 19.” American Institute of Steel Construction, Inc., Chicago, IL.

Schmidt, B. J., and Bartlett, F. M. (2002). “Review of resistance factors for steel: Data collection.” *Canadian Journal of Civil Engineering,* 29(1), 98-108.

Schneider, U., Diederichs, U., and Ehm, C. (1981). “ Effect of temperature on steel and concrete for PCRV’s.” *Nuclear Engineering and Design*, 67(2), 245-258.

SFPE (2000). “Guide to performance-based fire protection analysis and design of buildings.” Society of Fire Protection Engineers, Bethesda, MD.

SFPE (2004). “Engineering guide: fire exposures to structural elements.” Society of Fire Protection Engineers, Bethesda, MD.

Shin, K-Y., Kim, S-B., Kim, J-H., Chung, M., and Jung, P-S. (2002). “Thermo-physical properties and transient heat transfer of concrete at elevated temperatures.” *Nuclear Engineering and Design*, 212(1-3), 233-241.

Sultan, M. A. (1996). “A model for predicting heat transfer through noninsulated unloaded steel-stud gypsum board wall assemblies exposed to fire.” *Fire Technology,* 32(3), 239-259.

Stuckes, A. D., Tinker, J. A., and Simpson, A. (1986). “Effect of moisture on the thermal conductivity of lightweight aggregate concrete.” *Building Services Engineering Research and Technology*, 7 (1), 27-32.

Thomas, G. (2002). “Thermal properties of gypsum plasterboard at high temperatures.” *Fire and Materials*, 26(1), 37-45.

Tsantaridis, L. D., Ostman, B. A-L., Konig, J. (1999). “Short communication: Fire protection of wood by different gypsum plasterboards.” *Fire and Materials*, 23(1), 45-48.

Wainman, D. E. (1992). “The effect of load ratio on the limiting temperatures observed during two BS476: part 21 fire resistance carried out on 254x146mmx43kg/m universal beams protected with thick Vicuclad insulation board.” *Report No. SL/HED/R/S1199/18/92/C, British Steel.*

Whiting, D., Litvin, A., and Goodwin, S. E. (1978). “Specific heat of selected concretes.” *ACI Journal*, 75 (7), 299-305.

Wullschleger, L., and Wakili, K. M. (2008). “Numerical parameter study of the thermal behaviour of a gypsum plaster board at fire temperatures." *Fire and Materials*, 32(2), 103-119.

Figures

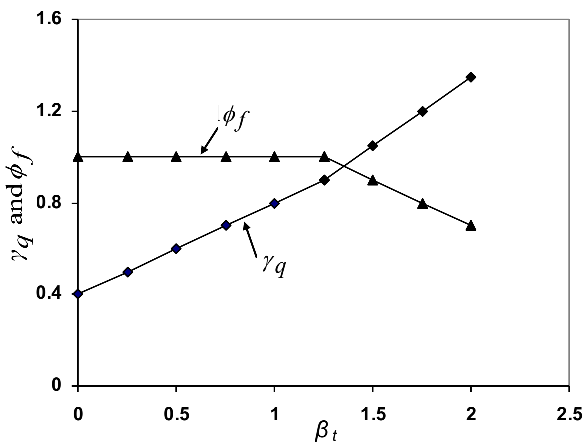

Fig. 1. Capacity reduction and fire load factors vs. target reliability index

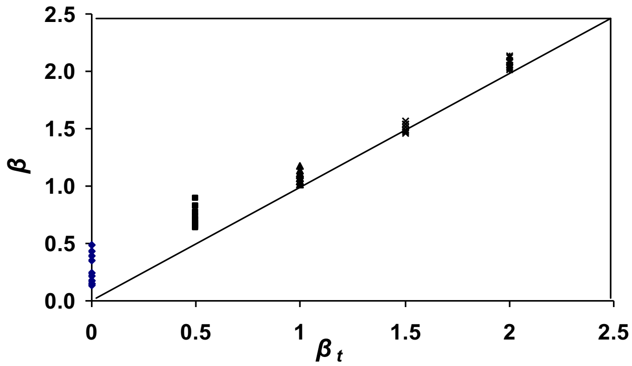

Fig. 2. Comparison of computed and target reliability index values

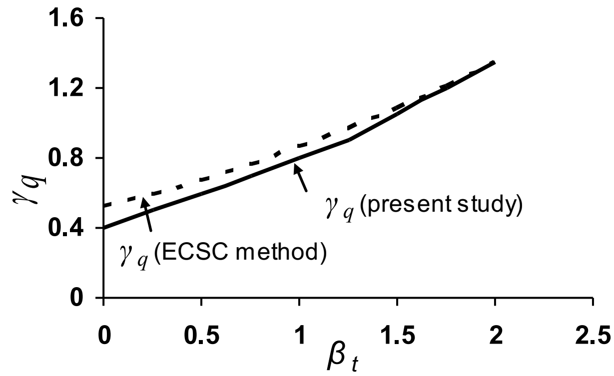

Fig. 3. Comparison of fire load factors with those obtained according to the ECSC method

Tables

Table 1. Mean, COV and distributions of fire design parameters

Variable	Mean	COV	Distribution
Arbitrary-point-in-time live load, L_{apt}	0.24*nominal	variable	Gamma
Dead load, D	1.05*nominal	0.100	normal
Fire load, q_f	564 MJ/m^2	0.62	Gumbel
Ratio of floor area to total area, A_f/A_t	0.192	0.23	lognormal
Opening factor, F_v	1*nominal	0.05	normal
Thermal absorptivity of normal weight concrete (NWC), b_{NWC}	1830 Ws$^{0.5}$/m^2 K	0. 094	normal
Thermal absorptivity of lightweight concrete (LWC), b_{LWC}	640 Ws$^{0.5}$/m^2 K	0. 107	normal
Thermal absorptivity of gypsum board, b_g	423.5 Ws$^{0.5}$/m^2 K	0.09	normal
Thermal absorptivity of a compartment having a 50/50 mix of NWC and gypsum board as boundaries, b_{mix}	1127 Ws$^{0.5}$/m^2 K	0.10	normal
Thickness of fire protection materials, d_i			
(1) spray applied materials	nominal+1/16 inch	0.20	lognormal
(2) gypsum board systems	nominal	0.05	normal
Density of fire protection materials, D_i			
(1) spray applied materials	307 kg/m^3	0.29	normal
(2) gypsum board systems	745 kg/m^3	0.07	lognormal
Thermal conductivity of fire protection materials, k_i, at temperature of 400-600°C			
(1) spray applied materials	0.187 W/m. K	0.24	lognormal
(2) gypsum board systems	0.159 W/m. K	0.28	lognormal

Note: The COV of the arbitrary-point-in-time live load depends on the tributary area (Ravindra and Galambos 1978) and is given as:

$0.82[1-0.00113(A_T-56)]$ for $56 \le A_T \le 336$ square feet

$0.56[1-0.0001865(A_T-336)]$ for $A_T > 336$ square feet