SDSS'Rio 2010 STABILITY AND DUCTILITY OF STEEL STRUCTURES
E. Batista, P. Vellasco, L. de Lima (Eds.)
Rio de Janeiro, Brazil, September 8 - 10, 2010

METHODOLOGY FOR RELIABILITY-BASED DESIGN OF STEEL MEMBERS EXPOSED TO FIRE

Shahid Iqbal and Ronald S. Harichandran

Dept. of Civil and Envir. Engineering, Michigan State University, East Lansing 48824-1226, USA
e-mails: iqbalsha@msu.edu, harichan@egr.msu.edu

Keywords: Reliability-Based Design, Steel Members, Fire Design, Statistics, Random Parameters

Abstract. *A general reliability-based methodology is proposed for developing capacity reduction and fire load factors for the load and resistance factor design (LRFD) 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, and their statistics, obtained from the literature and the analysis of raw data, are provided.*

1 INTRODUCTION

Performance-based codes allow use of engineering approaches for fire design of steel members instead of prescriptive approaches that are commonly used. For example, Appendix 4 of the 2005 AISC Specifications (referred to hereafter as "AISC Specifications") [1] now allows steel members to be designed against fire using room temperature design specifications and reduced material properties. A similar approach for design of steel members is included in Eurocode 3 [2]. 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 [1] allow using the same capacity reduction factors for fire design as those used for room temperature design. For example, $\phi_f = 0.9$ is suggested for beams. Most other codes suggest that a capacity reduction factor of 1.0 be used (e.g., in the Eurocode 3 [2], the partial safety factor 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 [3]. 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 [3]. However, limited work has been done to develop capacity reduction factors based on reliability analysis [4].

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. The AISC Specifications suggest that while calculating the steel and fire temperatures for fire design, due consideration should be given to the effectiveness of all active fire protection systems (sprinklers, smoke and heat detectors, etc.). The Commentary to the AISC Specifications [5] states that 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) (hereinafter 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 [6]. 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 reliability-based methodology is presented in this paper for developing capacity reduction and fire load factors for the load and resistance factor design (LRFD) of steel members exposed to fire. In addition, the uncertainties of design parameters that significantly affect the fire design are characterized. As an illustration, the statistics of the random variables and model errors derived in this study are used in the companion paper [7] for deriving capacity reduction and fire load factors for steel columns exposed to fire.

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

2 ENGINEERING APPROACH FOR DESIGN OF 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

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

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 steel temperatures are specified 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

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

where, D, L and S are nominal dead, live and snow loads, respectively, and T includes loads induced by the fire itself, especially due to restraint from the surrounding structure preventing thermal expansions.

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 insulation to slow down the rise of the steel temperature. The required thickness of insulation 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.

3 FIRE AND STEEL TEMPERATURES

The variation of fire temperature, T_f, with time can be estimated using a suitable mathematical model from the literature. In this study, the Eurocode parametric fire model modified by Feasey and Buchanan [8] is used to estimate the fire temperature under real fire scenarios.

Once the fire temperature variation with time is known, the temperature of steel members can be estimated through thermal analysis. Most design specifications such as the Eurocode 3 and AISC Specifications, 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 protected steel members can be written as [3]

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

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 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).

Equation (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 equation (4) was developed by Iqbal and Harichandran [9].

4 METHODOLOGY TO DEVELOP CAPACITY REDUCTION AND FIRE LOAD FACTORS

Capacity reduction and fire load factors can be developed by performing the following steps:

1. Obtain statistical parameters such as the mean, coefficient of variation (COV) and distributions of design parameters (e.g., yield strength of steel, cross-sectional area of steel, fire load, opening factor, etc.).
2. Select an appropriate performance function (design equation) for the structural member.
3. Characterize model errors (i.e., the professional factor) to account for the differences in the capacity calculated from the design equation and that measured in fire tests.
4. Select a target reliability index, β_t, which reflects the target probability of failure and is a relative measure of safety.
5. Calculate the capacity reduction and fire load factor through reliability analysis.

In the succeeding paragraphs, the above steps are elaborated in the context of steel members exposed to fire.

4.1 Statistics of random parameters

The parameters that significantly affect the fire design of steel members were chosen as random variables, and their means, COV, and distribution types are summarized in table 1. We analyzed raw experimental data to obtain the statistics of all parameters in table 1 except for the dead load, arbitrary-point-in-time live load and fire load. The statistics of the dead and arbitrary-point-in-time live loads were reported by Ellingwood [10] and Ravindra and Galambos [11], and the statistics of the fire load were taken from a survey of U.S. office buildings by Culver [12].

Bruls et al. [13] studied the variation of thermal conductivity of insulation at different temperatures. Although, thermal conductivity varies with temperature, they concluded that since the failure of structural steel members 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. In this study, a statistical analysis of thermal conductivity of gypsum board material in the temperature range of 400-600°C was performed to obtain the statistics shown in table 1.

4.2 Performance function for reliability analysis

Ellingwood [10] 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. The load effect W_f for reliability analysis may then be calculated as

$$W_f = E(c_D AD + c_L BL_{apt}) \tag{5}$$

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 [11].

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

$$R_f = P.f_R\left(f_1F_1,.......f_lF_l,k_1(t_sT_s)m_1M_1,.......k_k(t_sT_s)m_kM_k\right) \tag{6}$$

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 conductivity of normal weight concrete (NWC), k	1.747 W/m.K	0.171	normal
Specific heat of NWC, c_p	856 J/kg.K	0.062	normal
Density of NWC, ρ	2258 kg/m^3	0.069	normal
Thermal conductivity of lightweight concrete (LWC), k	0.372 W/m.K	0.199	Gumbel
Specific heat of LWC, c_p	826 J/kg.K	0.062	Gumbel
Density of LWC, ρ	1344 kg/m^3	0.069	normal
Thermal absorptivity of NWC, b_{NWC}	1830 Ws$^{0.5}$/m^2 K	0. 094	normal
Thermal absorptivity of LWC, b_{LWC}	640 Ws$^{0.5}$/m^2 K	0. 107	normal
Thermal absorptivity of gypsum board, b_q	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 [11] 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 where P, f_i, m_j, and t_s are the following non-dimensional random variables:

P = "Professional factor" reflecting uncertainties in the assumptions used to determine 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.

Using equations (5) and (6), the limit state equation for reliability analysis under fire conditions may be written as

$$g(\mathbf{X}) = R_f - W_f \tag{7}$$

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

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.

4.3 Model Error for Steel and Fire Temperatures

The maximum temperature of steel sections estimated using equation (4) differs from that measured in actual fire tests. 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. [14] 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 many possible real fire scenarios. Foster et al. [15] reported the temperature of four protected steel columns. In this test the fire load was 720 MJ/m^2 of the 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) [14], and the model error for the temperature of steel columns, t_{sc}, was characterized using the test data reported by Kirby et al. [14] and Foster et al. [15]. 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.

4.4 Probability of failure and target reliability index

CIB W 14 [16] 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. Safety factors depend on the selected target reliability index, β_t, which is related to the target 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 β_t by incorporating the effect of active fire protection systems was presented in the ECSC study [6]. The ECSC study also suggests 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, β_t was estimated for typical fire compartments in U.S. office buildings (ranging in floor areas from 25-500 m^2). It was found that it is reasonable to use β_t 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, β_t also varies for different design situations.

4.5 Determination of capacity reduction and fire load factors

4.5.1 Partial safety factors for each design parameter

For a normally distributed random design parameter, X, the partial safety factor resulting from the first-order reliability method (FORM) is given by [17]

$$\phi_X = \frac{m_X}{X_n}(1+\alpha_X \beta_t V_X) \tag{8}$$

where m_X, V_X, X_n, and α_X are the mean, COV, nominal value of X, and the direction cosine of the "design point," respectively. For non-normal random parameters (e.g., a lognormal distribution), the partial safety factor can be determined from equation (8) using the mean and standard deviation of the "equivalent normal variable" at the design point [17].

All the parameters in equation (8) are known from previous steps except the direction cosine of the design point which is obtained through reliability analysis.

4.5.2 Combination of partial safety factors into single capacity reduction factor

For convenience in design, the variability of all design parameters except for the fire load is accounted for through a combined capacity reduction factor instead of using a separate partial safety factor for each design parameter. The partial safety factors obtained through equation (8) for each individual design parameter except the fire load can be combined into a single capacity reduction factor. The fire load is a major parameter in fire design, and uncertainty associated with it 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 AISC specifications recommend that the fire load be reduced by 60% if a reliable sprinkler system is installed. This recommendation also motivated use of a separate safety factor for the fire load. This fire load factor is to be applied to the fire load used in describing the design fire (or time-fire temperature curve).

4.5.3 Optimal capacity reduction and fire load factors

The capacity reduction and the fire load factors will vary for each design situation. For ease of design, it is desirable to have a single optimal capacity reduction factor applicable to all design situations. In addition, for fire design of steel members, the AISC Specifications recommend using dead and live load factors of 1.2 and 0.5, respectively. Therefore, the optimization procedure described in NBS 577 [17], and summarized below can be used to develop optimal capacity reduction and fire load factors corresponding to dead and live load factors of 1.2 and 0.5, respectively.

The nominal capacity for each design situation based on the FORM is

$$R_n^{II} = \frac{1}{\phi_{f,i}^{II}} \sum_i \gamma_i^{II} Q_{n,i} \tag{9}$$

where γ_i^{II} and $\phi_{f,i}^{II}$ are the load and resistance factors for each design situation, and $Q_{n,i}$ are the nominal values of dead and live loads. The nominal capacity corresponding to the recommended dead and live load factors is

$$R_n^{I} = \frac{1}{\phi_f} \sum_i \gamma_i Q_{n,i} \tag{10}$$

where the γ_i are 1.2 and 0.5 for dead and live load, respectively. ϕ_f is the optimal capacity reduction factor, and its value in equation (11) can be selected by minimizing the objective function

$$\varepsilon(\phi_f) = \sum \left[R_{n,j}^{II} - R_{n,j}^{I}\right]^2 p_j \tag{11}$$

where p_j is the weight assigned to the jth design situation. In this study, each design situation was assigned the same weight.

5 CONCLUSION

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.

REFERENCES

[1] AISC. “Load and resistance factor design specifications for structural steel buildings”, American Institute of Steel Construction, Inc., Chicago, Illinois, 2005.

[2] EN. “Eurocode 3: Design of steel structures- Part 1-2: General rules-Structural fire design”, BS EN 1993-1-2, 2005.

[3] Buchanan, A. H. *Structural Design for Fire Safety*, John Wiley & Sons Ltd., New York, 2001.

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

[5] AISC. “Commentary on the load and resistance factor design specifications for structural steel buildings”, American Institute of Steel Construction, Inc., Chicago, Illinois, 2005.

[6] European Coal and Steel Community (ECSC). “Natural fire safety concept, valorization project”, Product-Structural Dept. ARBED-Research Centre L-4009 ESCH/ALZETTE, Luxembourg, 2001.

[7] Iqbal, S. and Harichandran, R. S. “Capacity reduction and fire load factors for steel columns exposed to fire”, *Proc. of SDSS’Rio 2010 Stability and Ductility of Steel Structures*, Rio de Janeiro, Brazil, 2010.

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

[9] Iqbal, S. and Harichandran, R. S. “Reliability-based design specifications for simply supported steel beams exposed to fire”, *Proc. of 10th International Conference on Structural Safety and Reliability*, Osaka, Japan, 2009.

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

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

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

[13] Bruls, A., Cajot , L. G. and Franssen, J. M. "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, 1988.

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

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

[16] CIB W14. "Design guide for structural fire safety". *Fire Safety Journal*, **10**(2), 81-137, 1986.

[17] Ellingwood, B. Galambos, T. V., MacGregor, J. G. and Cornell, C. A. "NBS special publication 577: Development of a probability based load criterion for American National Standard A58", National Bureau of Standards, Ernest Ambler, 1980.