University of Canterbury Fire Engineering Conference
C Manning (Ed.)
Christchurch, New Zealand, February 27, 2015

ASSESSMENT OF FIRE PERFORMANCE OF STEEL I-SECTION COLUMNS WITH WEB INFILLED CONCRETE

Christian B. Manning *

* e-mail: cbm38@uclive.ac.nz

Keywords: Web Infilled Concrete, Steel I-Section Columns, Fire.

Abstract. This paper determines whether the increases in fire resistance gained by concrete filling I section steel columns can be realistically determined by the application of the bare steel fire resistance provisions of NZS 3404 [1] with a simple modification of the section factor. This determination was made by comparing the predicted fire resistance determined with the program SAFIR and comparing the results to a number of Standard Fire tests on these columns. The results show that the approach is realistic and that a simple modification to the section factor improves the accuracy of the bare steel fire resistance provisions of NZS 3404 equations.

1 INTRODUCTION

According to Buchanan [2] steel is a frequently used construction material owing to its high strength and stiffness relative to its weight and versatility as a construction material. However, when exposed to fire, its strength and stiffness reduce, leading to possible deformation and failure, as the steel temperature increases. Columns also in practice have a lower fire resistance than what they would have in the Standard Fire test, as axial loads do not increase in the Standard Fire test as they do during a real building fire due to the effects of restrained thermal expansion and the limited ability to shed loads between the columns.

NZS 3404[1] has a chapter dedicated to steel elements requiring a FRR. This chapter contains simple formulae which determine the time at which an element is unable to continue to sustain the structural fire severity generated by exposure to the Standard Fire.

As the temperature of a member increases its strength decreases. The lower the level of load applie to the member the higher the temperature that may be achieved by the member before failure. The calculation of the limiting temperature is expressed in NZS 3404.

The relationship of how steel temperature varies with steel mechanical properties is outlined in NZS 3404 Clause 11.4. The two varying mechanical values of steel elastic modulus and yield stress vary with temperature and their values decrease as the steel temperature increases. The values change little up around 215 °C after which they reduce in an approximately linear fashion down to zero as the steel temperatures increase towards 900-1000 C.

The formula that determines the time at which the limiting temperature is reached is dependent upon the applied loading and the steel section factor (SF). The SF which is denoted as Hp/(Ax7.85) is the exposed surface area to mass ratio (in square metres/tonne) of the steel element and influences the rate in a column with no web infill. The possibly for increased use of this method of partial protection of bare structural steel in buildings means that a fundamental examination of the simple formula provided in NZS 3404[1] is required.

2.1 Rise in temperature in unprotected steel

In NZS 3404 the time (t) when the limiting temperature (Tl) is reached is calculated for four sided exposure of unprotected steel members (subject to the Standard Fire test exposure) as follows:

$$t = -4.7 + 0.0263\,T_l + \frac{0.231\,T_l}{SF} \qquad (1)$$

SF = section factor Hp/(A x 7.85) m^2/tonne, 2 < section factor < 35 m^2/tonne.

T_l = limiting temperature, in degrees Celsius, 500 C < steel temperature < 850 C

The temperature range given in NZS 3404 is applicable to beams but may not be applicable to columns because of the effects of structural restraint against expansion which increases the demand on t column compared with that in a Standard Fire test. For that reason, design guides such as Spearpoint [5] impose lower upper limits on the calculated column limiting temperature of up to 600 C

In NZS 3404 it describes how the Period of Structural Adequacy (PSA) is determined using the following 3 methods:

1. By using formulae expressed in NZS 3404 to determine the time when the limiting temperature is reached. First calculating T_l = 905 – 690 r_f where rf is the ratio of the design action on the member under the design load to the design capacity.
2. The direct application of a single Standard Fire test; or
3.

4 NUMERICAL MODEL ANALYSIS PROCEDURE

The thermal and structural analyses in this paper

Base fully fixed

Figure 4: Loading on Steel Columns

The column members modelled were 3 metres with the same physical and section properties as the actual tests. Each column support was modelled with a fixed support for translation and rotation, except for free to axially elongate so as to allow for thermal expansion to occur during the fire heating process.

4.4 Standard Fire Modelling

The Standard ISO 834 heating curve from BS EN 1363-1 [9] has been used in the SAFIR analysis and was used in the furnace tests. The time-temperature curve of the Standard Fire is calculated in Equation 2.

T = 34P <<io9[(Ba)(p)-199[(B0)584(u)-6(rs)7(P <<)11506iT t=

T =

Figure 5: Rise in web temperature

The test temperatures for the web were measured at the column-height centre and mid-depth in the flange at a quarter of the way along its length.

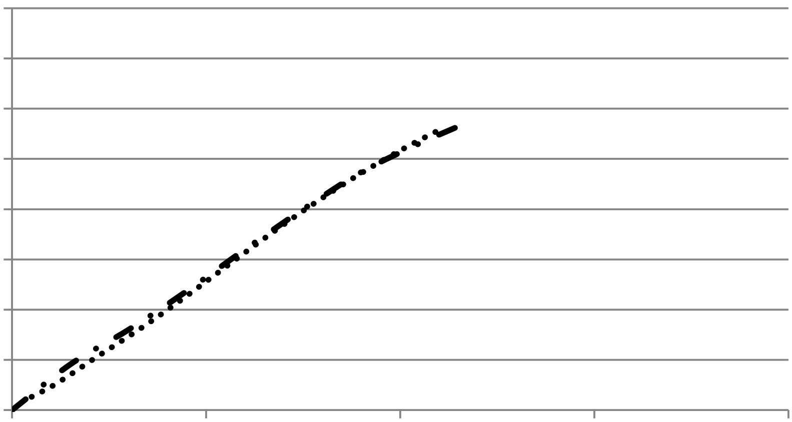

Figure 6: Rise in flange temperature

A significant reduction in temperature was observed in the webs of the steel columns due to blocking in of the flanges. Over-prediction of the flange temperatures was observed in SAFIR due to the values used of coefficient of absorption (depth of surface absorption) and the emissivity (amount of incident radiation absorbed/ reflected).

Table 2: Comparative test results

Column size	Load (kN)	r_f	Time to failure (minutes)				Comparative results (%)	
			NZS 3404 SNZ (1997)				SAFIR : NZS 3404	
			Unmodified Equation (1)	Modified Equation (4)	SAFIR	Test	Unmodified Equation (1)	Modified Equation (4)
150UC23	381	0.71	13	17	22	23	0.59	0.77
200UC46	811	0.62	19	26	30	30	0.63	0.87
200UC52	550	0.38	28	38	38	38	0.74[2]	1.0
200UC52	916	0.62	19	26	35	36	0.54	0.74

Better agreement for comparisons for time to failure are seen in the lightly loaded sections with smaller rf comparatively than in the larger cross sections, indicating that the actual relationship is more complex than presented by the NZS 3404 [1] Clause 11.6 provisions.

,Q WKH ODVW WHUP LQ WKH 1=6 ³WLPH WR IDLOXUH´ (TX particular for the heavy mass sections and when most heavily loaded.

7KH DXWKRU VXJJHVWV D ³PRGLILHG´ 1=6 (TXDWLRQ V SF term, as in Equation (4).

$$t = -4.7 + 0.0263\,T_l + \left[\frac{0.231\,T_l}{\boldsymbol{0.6} \times SF}\right] \tag{4}$$

This factor is a best fit after having looked at a number of variables. Its purpose is to reduce conservatism without making the results un-conservative and given the relative nature of this approach defining this factor to more than 1 decimal place is not appropriate.

2 If Spearpoint (2008) limiting temperature guidance of 600 °C were applied, the ratio of comparative agreement NZS 3404/ SAFIR would decrease from 74% to 68%.

Figure 8: Time to failure comparisons

Applying this factor would improve the comparison with the NZS 3404 [4] equations giving improved correlation between SAFIR results compared with the modified equation shown in Figure 8.

The justification for modifying the NZS 3404 Equation (1) is that, as per the original equations, this finding is a curve fit of experimental results. While the linear line indicated on Figure 8 is just a trend fit given in reality the parameters change in a discrete fashion not in a continuous change.

8 CONCLUSIONS

The author suggests improving the accuracy of the NZS 3404 equation