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

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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.

Similar to columns, beams in buildings also have axial and rotational restraint at their connections. But in beams, during the heating phase, they undergo downwards deflection (towards the fire) and thermal expansion. The combination of these two effects largely cancels out change in length in plan, meaning relatively small axial load demands on their connections in the heating phase, while their rotational restraints develop negative end moments, increasing their fire resistance, providing that their local or member instability doesn't become severe. Therefore beams in the heating phase in a real building typically have a greater fire resistance than in the Standard Fire test. In their cooling phase, high tension due to pull-in can lead to local connection failure, but this is suppressed by suitable detailing, which is also required for ductility in earthquakes.

For columns there are applications such as car parks where the structural fire severity may be only slightly greater than the resistance offered by the bare steel and there may be a way to achieve the necessary resistance, without the need of protecting the bare steel column's entire heated perimeter.

With the publication of C/VM2 [3], there is expanded scope for the use of such I-section partially protected columns, as this will give up to a 30 minute Fire Resistance Rating (FRR) for typical columns (which is often all that is required). This paper investigates this little used option of partial protection by the use of concrete block infill between the flanges and the web of structural steel I-section columns. Also, although not considered further in this paper, this option enhances the robustness of the I-section column against impact loads such as those from vehicles, with very little additional cost.

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 applied 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 to 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 at which the temperature increases. A bare unprotected steel section has a large SF due to its large surface area to mass ratio and this results in a faster temperature rise compared to the same size section that has block infill in its webs. When an I-section column has concrete infill between the flanges it greatly reduces the SF.

2 BACKGROUND

In the NZ 3404, there is a formula provided for 4-sided exposure of bare steel columns but in fire large columns are required to get the 30 minute rating and these sections are larger than otherwise needed for non-fire cases. Therefore there is potential for lighter sections with reduced SF due to their infill. The purpose of this research has been to see if this was or was not valid. To see if applying a single modification could be made if the formula found from investigation is conservative.

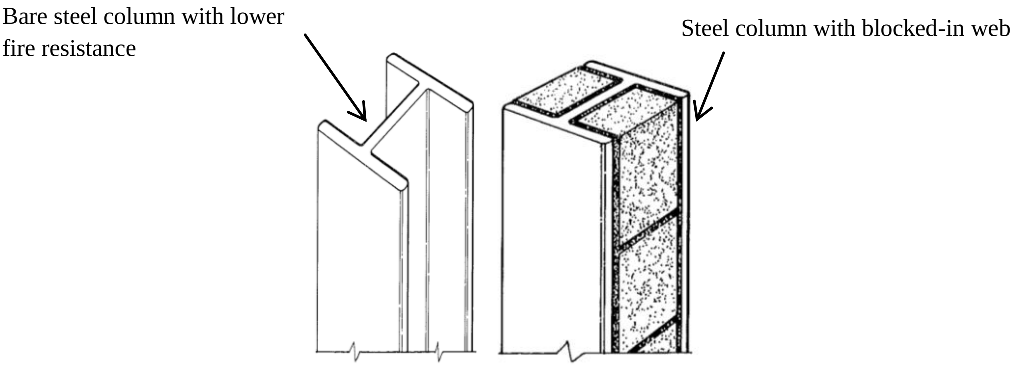

Figure 1: Columns with enhanced fire resistance [4]

The formula applies a limiting temperature to the entire cross-sectional area to determine the time at which it is reached on exposure to the Standard Fire. The formula for limiting temperature is derived from the equations that relate to the variation of steel yield stress as temperatures rise above 215 °C.

In this paper a comparative study has been undertaken by the author using finite element analysis (FEA) to compare and assess the accuracy of the formula in NZS 3404. The reason FEA is undertaken is to verify whether the NZS 3404 formula for unprotected bare steel work with Hp/(Ax7.85) adjusted is conservative. The reason for this could lie in the significantly reduced web temperature compared to that 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:

$$t = -4.7 + 0.0263\, T_l + \left[\frac{0.231\ T_l}{SF}\right] \tag{1}$$

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 the 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. By structural analysis using the variations of the mechanical properties of steel with temperature confirmed by test data.

In this paper results from the NZS 3404 formulae are compared with those from the SAFIR analysis and from actual fire tests compiled by Wainman and Kirby [6]. The steel in the tests performs in fire the same as New Zealand and Australian steels due to having the same grade, metallurgical and mechanical properties.

3 STEEL FIRE RESISTANCE PROVISIONS IN NZS 3404

The paper provides new computer modelling results that characterises the thermal and structural response of block infilled steel I-section columns when subject to compression and heating. It considers and compares the results obtained from all three of the methods listed in NZS 3404.

It addresses the various uncertainties and limitations surrounding the use of NZS 3404 equations for such partially protected column members. In addition it provides recommendations and guidance where it does not currently exist.

This paper presents studies of the following:

- Summarises the different PSA determination methods present in Chapter 11 of NZS 3404 for partially protected columns.
- Uses the computer program SAFIR as a finite element software program to determine the PSA and compare the modelling results with the physical test results compiled by Wainman and Kirby and examines how closely they match.
- Looks at the rationality of the equations present in NZS 3404 and compares the results of these equations with those derived from the SAFIR simulations.
- Makes proposals based on the findings in this paper, for changes to NZS 3404.

4 NUMERICAL MODEL ANALYSIS PROCEDURE

The thermal and structural analyses in this paper are conducted with the use of the two dimensional non-linear finite element computer program; SAFIR as described by Franssen [7]. SAFIR is used for the analysis of complex structural members under fire conditions.

The technical approach for this analysis involved the use of modelling and numerical analysis using FEA software in which the member is divided up into a series of segments and each segment is represented by a cross section. Within the FEA program there is a ‘thermal’ and ‘structural’ part to the analysis, a brief description of these follows.

4.1 Thermal analysis

The ‘thermal’ analysis is used to predict the temperature distribution inside the different column cross-sections being studied when exposed to fire.

For each column there is the modelling assumption made that there is no heat transfer along the axis of the column so that the same thermal cross profile will apply along its length. Starting with creating the model in 2D there are steps explained in the SAFIR user manual. Once a two-dimensional model is created, input files can then be generated for a 3D structural analysis to enable torsional effects to be considered.

For the thermal analysis to be performed, the thermal properties such as the conductivity and specific heat for both the concrete and steel materials are defined and the cross-section discretised into smaller regular shaped elements. Figure 2 shows how the cross section is then divided up into a mesh which allows SAFIR to analyse how the materials react to the changes in temperature throughout their depth when exposed to four-sided external heating.

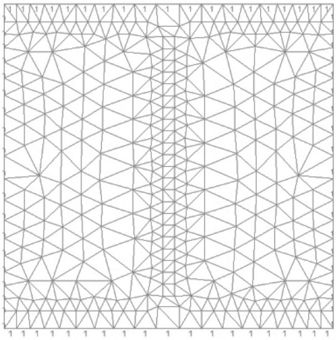

Figure 2: Diamond thermal mesh representation output screen

4.2 Thermal results

Once the FEA runs are complete, the results of the thermal analysis are reviewed using other programs that are part of the suite of companion programs that have been devised for use with SAFIR. One post-processor program is called Diamond 2012. By using the Diamond post-processing program it enables the thermal analysis runs developed using FEA to also be reviewed graphically. The Diamond program allows the viewing of a slice through the section which changes over time. In Figure 3 it can be seen how the temperature in the cross section changes during heating.

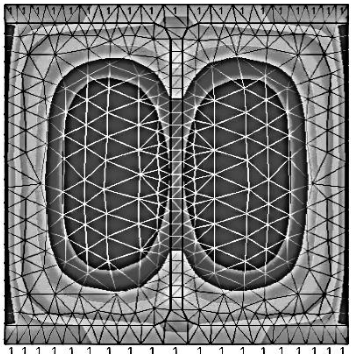

Figure 3: Diamond thermal post-processor output screen

Once the thermal analysis is complete the entire member is then modelled. This creates an element that has a discrete length and this allows support conditions and loads to be applied and from this 'structural' analysis the time to failure is predicted.

4.3 Structural Analysis

The different standard section sizes and loading were used in modelling were as in the actual fire tests Wainman and Kirby [6] complied in Table 1.

Table 1: Formulae and test input values

Column size	**150UC23**	**200UC46**	**200UC52**	**200UC52**
Applied Load N_c[1] (kN)	381	811	550	916
$\emptyset$ N_C (kN) AISC [8]	483	1180	1320	1320
$\emptyset_{fire}$ $N_{C\ MIN}$ (kN)	537	1311	1467	1467
Area A[1] (mm^2)	2980	5900	6660	6660
b_f[1] (mm)	151	202	204	204
t_f[1] (mm)	7	10.5	12.5	12.5

[1] Loads and geometric properties from Wainman and Kirby [6]

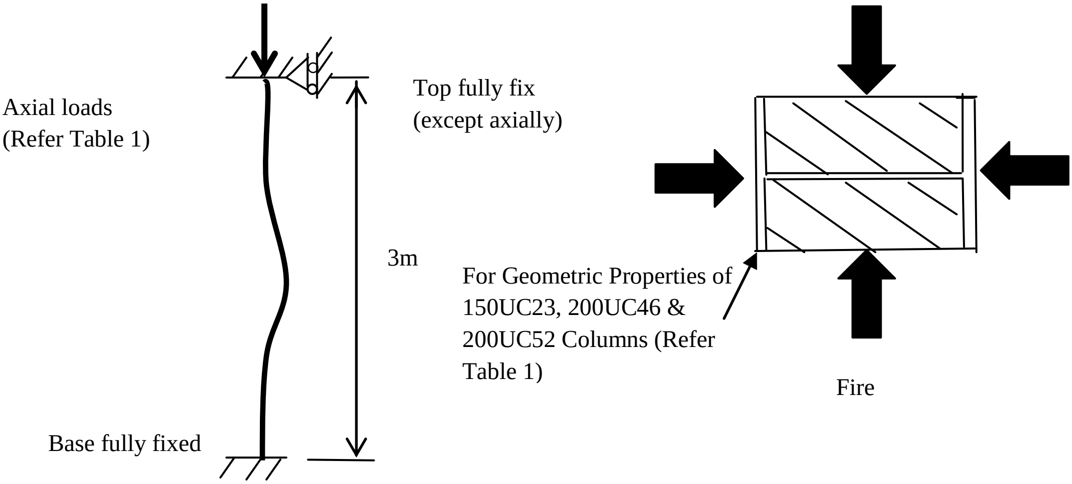

Figure 4: Loading on Steel Columns

The column members modelled were 3 metres tall 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.

$$T = 345 \times \log_{10} (8t + 1) + 20 \qquad (2)$$

Where T is the temperature (°C) and t is the time (minutes).

4.5 Failure Criteria

The SAFIR columns were subjected to the same failure criteria as the furnace test columns as in BS 476-20 [10] when no longer able to support the axial load either by:

(a) A deflection of greater than L/20 or when
(b) A rate of deflection greater than in Equation 3.

$$R = L2/(9000 \times d) \qquad (3)$$

Where R = rate of deflection in mm/min
L = the span of the column in mm
d = the depth from the compression face to the tension zone in mm

5 NUMERICAL MODEL RESULTS

5.1 Thermal results

The thermal results for each member showing the changes in temperature in the webs and flanges are plotted in Figures 5 and 6.

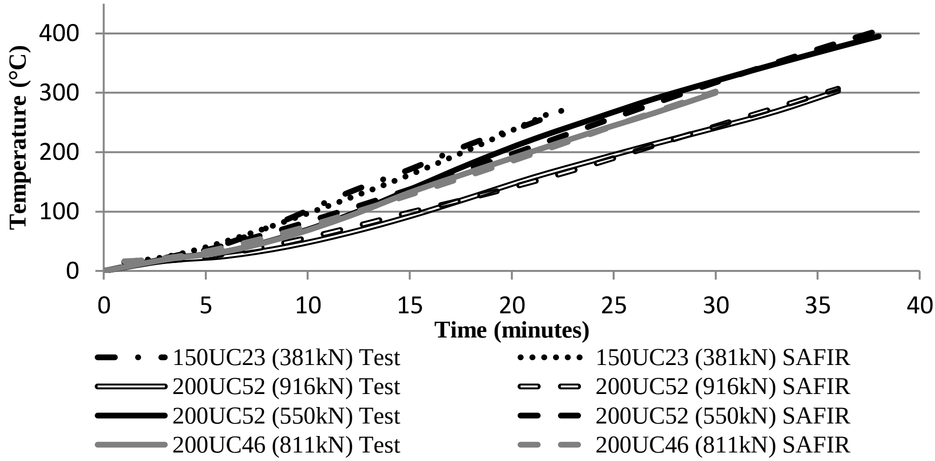

Figure 5: Rise in web temperature

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

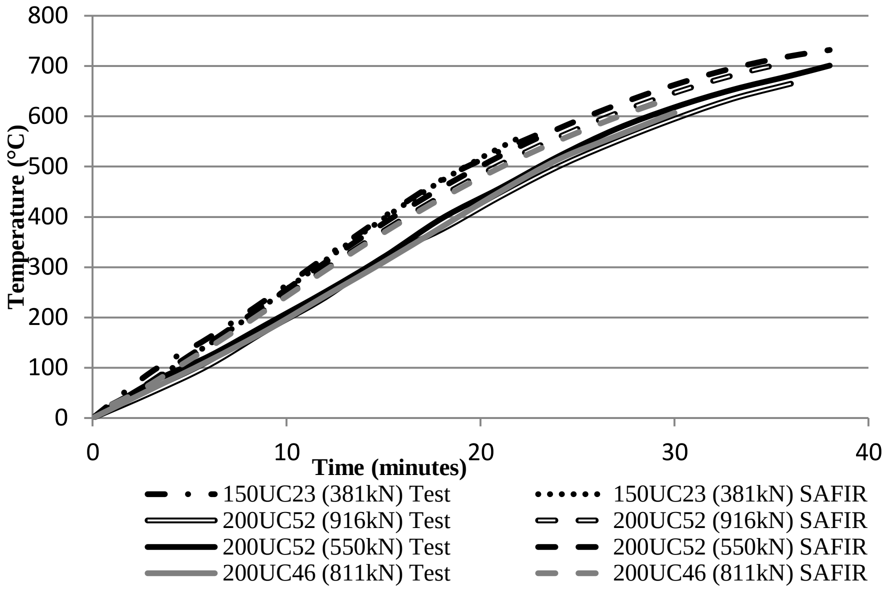

Figure 6: Rise in flange temperature

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).

Purkiss [11] describes how shielding of the web and inner flanges induces thermal gradients in the steel members, which from this assessment of the temperature gradients occurring in the heated steel should lead to more accurate results compared to that predicted in the NZS 3404 [1] equations.

5.4 Structural results

The structural results for each member showing the changes in vertical displacement and time to failure are plotted in Figure 7 which shows the deflection changes due to elongation and the time to failure.

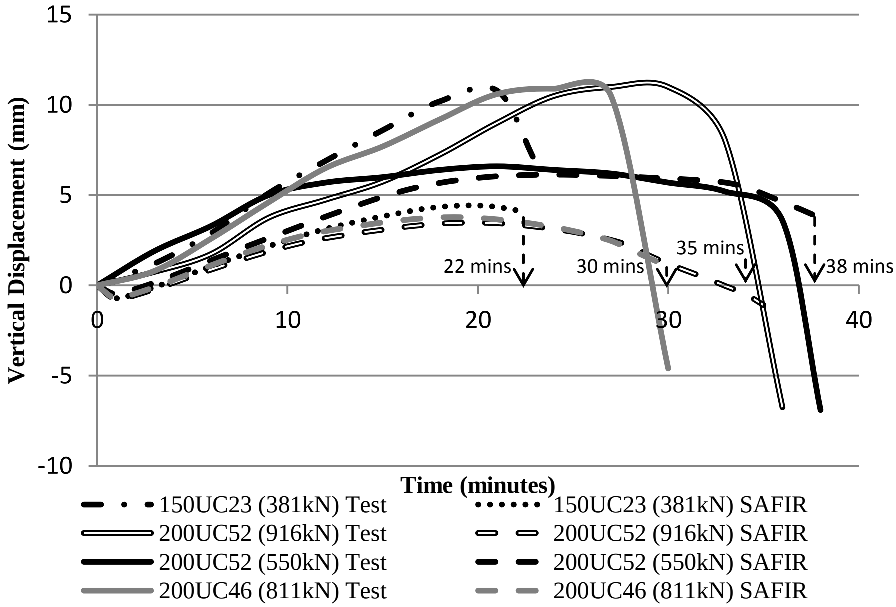

Figure 7: Vertical displacements and time to failure (given in Table 2)

7 COMPARISON OF FEA RESULTS AND STEEL FIRE RESISTANCE PROVISIONS OF NZS 3404

The analysis for temperature compares well with the actual fire tests but due to a function of the way SAFIR models failure the deflection plots terminate more abruptly when modelled than in the actual fire tests. This is because when approaching failure, due to numerical instability in SAFIR, the analysis stops at the very beginning at the point when a rapid increase in deflection (indicating failure) occurred in the actual member failures.

The likely reason why the deflections in SAFIR were greater than those in the tests was due to the way in the tests (due to the gap in the concrete at the top of the columns) the concrete to steel interface bond broke when subject to heavy axial loads. Whereas deflection agreed in the plot when lightly loaded as reduced axial loads caused the bond to be less broken.

Comparisons in Table 3 with the analysis and actual test results show that the NZS 3404 equations are conservative.

Table 2: Comparative test results

Column size	Load (kN)	r_f	Time to failure (minutes)				Comparative results (%)	
			NZS 3404 SNZ (1997)		SAFIR	Test	SAFIR : NZS 3404	
			Unmodified Equation (1)	Modified Equation (4)			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.

In the last term in the NZS 3404 "time to failure" Equation (1) the SF does not decrease fast enough, in particular for the heavy mass sections and when most heavily loaded.

The author suggests a "modified" NZS 3404 Equation (1) to remedy this with a factor of 0.6 added to the SF term, as in Equation (4).

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

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, then ratio of comparative agreement NZS 3404/ SAFIR would decrease from 74% to 68%.

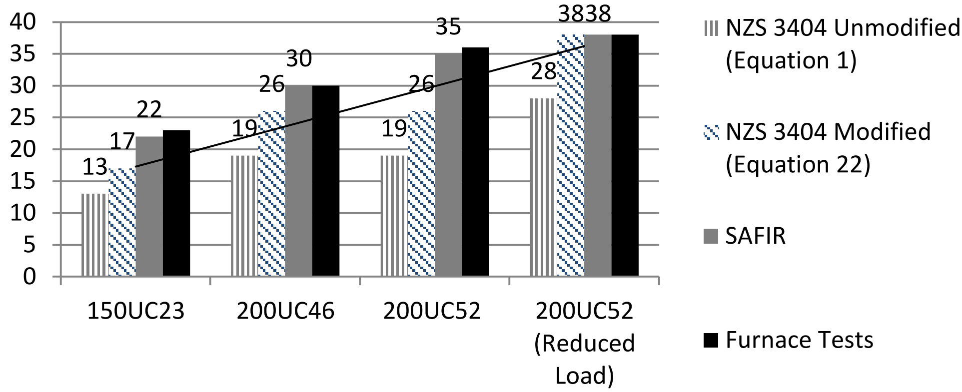

Figure 8: Time to failure comparisons

Applying this factor would improve the comparison with the NZS 3404 [1] 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 11.6.2 for application to unprotected I-section columns with web infilled concrete block, through adding a simple modification to the SF in the equations.

The comparison with the NZS 3404 equations giving improved correlation with SAFIR with the modified equations is shown in Figure 8.

This research showed that the effects of partial shielding of the web and exposed flanges with no additional protection has the effect of inducing thermal gradients in the UC sections therefore allowing a redistribution of carrying capacity from the hotter to the cooler parts of the steel member. This reduces the applied stresses, which has the effect of increasing the inherent fire resistance of the steelwork.

The use of block work infill will also enhance the robustness of the I-section column against impact loads such from vehicles with very little additional cost, whilst beyond the scope of this project this could be an additional beneficial effect since re-straightening open steel columns by heating them to high temperatures above 650 °C risks increased local buckling due to the introduction of constrained axial expansion forces during the heating process.

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