Flexural-Torsional Buckling Behaviour and Design of Cold-formed Steel Compression Members at Elevated Temperatures

Shanmuganathan Gunalan, Yasintha Bandula Heva and Mahen Mahendran

Abstract:

Current design rules for the member capacities of cold-formed steel columns are based on the same non-dimensional strength curve for both fixed and pinned-ended columns at ambient temperature. This research has investigated the accuracy of using current ambient temperature design rules in Australia/New Zealand (AS/NZS 4600), American (AISI S100) and European (Eurocode 3 Part 1.3) standards in determining the flexural-torsional buckling capacities of cold-formed steel columns at uniform elevated temperatures using appropriately reduced mechanical properties. It was found that these design rules accurately predicted the member capacities of pin ended lipped channel columns undergoing flexural torsional buckling at elevated temperatures. However, for fixed ended columns with warping fixity undergoing flexural-torsional buckling, the current design rules significantly underestimated the column capacities as they disregard the beneficial effect of warping fixity. This paper has therefore recommended the use of improved design rules developed for ambient temperature conditions to predict the axial compression capacities of fixed ended columns subject to flexural-torsional buckling at elevated temperatures within AS/NZS 4600 and AISI S100 design provisions. The accuracy of the proposed fire design rules was verified using finite element analysis and test results of cold-formed lipped channel columns at elevated temperatures except for low strength steel columns with intermediate slenderness whose behaviour was influenced by the increased nonlinearity in the stress-strain curves at elevated temperatures. Further research is required to include these effects within AS/NZS 4600 and AISI S100 design rules. However, Eurocode 3 Part 1.3 design rules can be used for this purpose by using suitable buckling curves as recommended in this paper.

Keywords: *Cold-formed steel columns, elevated temperatures, flexural-torsional buckling, finite element modelling, design rules.*

Corresponding author's email address: m.mahendran@qut.edu.au

1. Introduction

Cold-formed steel members are becoming increasingly popular in the construction industry due to their superior strength to weight ratio and ease of fabrication as opposed to hot-rolled steel members. They are often subject to axial compression loads in a range of applications (Figure 1). These thin-walled members can be subject to various types of buckling modes, including flexural-torsional buckling. Many design standards provide guidelines for columns subject to flexural-torsional buckling at ambient temperature. However, there are no specific design guidelines for cold-formed steel members at elevated temperatures. Hence extensive research efforts have gone into the many investigations addressing the buckling behaviour of cold-formed steel columns at ambient and elevated temperatures [1-9].

In order to investigate the flexural-torsional buckling behaviour of cold-formed steel columns, an experimental study was carried out on lipped channel sections at ambient and uniform elevated temperatures by Bandula Heva and Mahendran [5]. Test cross-section dimensions and column lengths were selected based on a number of preliminary analyses using finite strip analysis (CUFSM [10]) and finite element analysis (ABAQUS [11]) programs so that flexural-torsional buckling govern the member behaviour. Tests of 1800 mm and 2800 mm long columns were conducted at ambient and elevated temperatures up to 700 °C (20°C, 200 to 700 °C at 100 °C intervals). In their experimental study, the specimen length was limited to 2800 mm to obtain flexural torsional buckling.

In this research, nonlinear finite element analyses of cold-formed steel columns subject to flexural-torsional buckling were undertaken using ABAQUS [11]. Reduced mechanical properties [12,13] were used with appropriate boundary conditions. The results of finite element analyses were compared and validated with test results reported in Bandula Heva and Mahendran [5] for fixed ended cold-formed steel columns. The overall aim of this research is to investigate the accuracy of current ambient temperature design rules in determining the strengths of concentrically loaded cold-formed steel columns with both fixed and pinned ends at elevated temperatures subject to flexural-torsional buckling using the validated finite element model. Hence a detailed parametric study was then carried out to understand the effects of different parameters that affect the flexural-torsional buckling behaviour of cold-formed steel compression members at ambient and uniform elevated temperatures. The selected lipped channel section columns in this research were not subject to flexural buckling. Three different steel grades and thicknesses were considered in this numerical study to

investigate the effect of using low and high grade steels. Three different section sizes were also considered in this study with varying column lengths. The accuracy of using the current ambient temperature design rules in Australia/New Zealand, North American and European cold-formed steel standards [14-16] in the fire design of cold-formed steel compression members subject to flexural torsional buckling was assessed by employing appropriately reduced mechanical properties. The results obtained from this study were compared with the predicted ultimate loads from the current cold-formed steel design standards, based on which the accuracy of current design rules for pinned and fixed ended cold-formed steel compression members subject to flexural-torsional buckling at elevated temperatures was investigated. This paper presents the details of this research study and the results.

2. Experimental Study

Bandula Heva and Mahendran [5] investigated the behaviour and strength of cold-formed steel lipped channel columns at both ambient and elevated temperatures. Three grades and three thicknesses, G550-0.95 mm, G450-1.90 mm and G250-1.95 mm, were used in their experimental study to represent light gauge cold-formed steel domain. The most common cold-formed steel column section is the lipped channel section. Figure 2 shows a typical buckling plot of lipped channel sections as obtained from a finite strip analysis program CUFSM. Specimen lengths were selected by avoiding the local and distortional buckling regions. The selected sections from CUFSM analyses were further analysed using ABAQUS to ensure the occurrence of the desired flexural-torsional buckling mode. Table 1 shows the details of test specimens while Table 2 presents the mechanical properties, Young's modulus and yield stress, at ambient and elevated temperatures. Table 3 gives the measured cross-sectional dimensions and lengths of tested specimens.

Both ambient and elevated temperature tests of long columns were carried out inside an electric furnace. The test set-up consisted of a reaction frame, furnace, control system, loading set-up and hydraulic loading system. In the case of compression tests with long columns subjected to global buckling, end conditions are very important. Since it is difficult to achieve an ideal pin-end condition inside the furnace, all the tests were carried out using fixed-end supports. To achieve fixed-end supports, special end plates were made to fit into the ends of specimens and grouted after inserting the specimen. Ceramic fiber insulation packing was used between the plate on the loading shaft and the end plate of the specimen to prevent heat loss. Test program includes two series of tests based on the length of test specimens (1800

mm and 2800 mm long columns). Figures 3 (a) and (b) show a test column from each series located inside the electric furnace. Two segments of the furnace were used for shorter columns while three segments were used for longer columns. Once the specimen temperature reached the target temperature, the specimen was allowed another 10 minutes to ensure a uniform temperature in the column before loading. During the heating phase, special care was taken to monitor the load applied to the specimen due to thermal expansion effects. Once a steady state was reached, load was applied slowly using the hydraulic jack until failure. Stainless steel cables were used to connect LVDTs, which were kept outside the furnace and the specimen to measure its deflections at mid-height. Axial shortening was measured using a 50 mm LVDT attached to the bottom loading shaft. Ambient temperature tests were also conducted inside the furnace while keeping the doors open.

As expected, all the specimens failed in flexural-torsional buckling except the G550-1800-300 specimen, which showed interaction of flexural-torsional buckling and local buckling. Since the elevated temperature tests were conducted inside the furnace, the deflected shape could not be observed during the test. The failure pattern of specimens tested at elevated temperatures was determined from the load-deflection curves. Larger deformations, particularly the out-of-plane deflection and twisting, were observed with the tests at higher temperatures. Larger deformations were due to reduced elastic modulus at elevated temperatures and thus lower flexural and torsional rigidities. From the tests, axial compression load versus axial shortening and out of plane deflection curves, and the ultimate failure loads (P_{ult}) were obtained (Table 3).

3. Finite Element Modelling

In this research ABAQUS was used in the finite element analyses (FEA) of cold-formed steel columns subject to axial compression loads. Finite element models were first developed to simulate the tested lipped channel column specimens (Table 3). They were G550-0.95-1800, G550-0.95-2800, G450-1.90-1800, G450-1.90-2800, G250-1.95-1800 and G250-1.95-2800 series specimens at temperatures in the range of 20 to 700°C. The measured dimensions of test specimens including their base metal thicknesses given in Table 3 were used in the analyses. Suitable element type and size were selected considering the accuracy and economy of simulations. Appropriate boundary conditions were used to simulate the observed flexural-torsional buckling behaviour in the tests. Imperfections and residual stresses were also

included in the analyses. Elevated temperature tests were simulated by using appropriately reduced mechanical properties.

3.1. Element Type and Size

Thin cold-formed steel plates can be represented with shell elements such as S4, S4R, S4R5 and S8R5. These element types were considered in the simulation of G450-1.90-2800-20 specimen in order to select the most suitable element type. Ultimate compression loads obtained from FEA were compared with test results. This comparison showed that all the element types give similar ultimate compression loads. Therefore S4 element type was selected for flexural-torsional buckling simulations because it is a fully integrated, general-purpose shell element available in ABAQUS standard version.

Element size of the model is also an important factor in FEA. Finer mesh gives more accurate results. However, it needs more memory and processing time. Hence a finer mesh does not give the most economical simulation. Therefore a convergence study was carried out for G450-1.90-2800-20 specimen with various mesh sizes from 15 mm x 15 mm to 3 mm x 3 mm to find the optimum mesh size. It was found that 5 mm x 5 mm mesh size gives the optimum results with acceptable processing time and memory requirements.

3.2. Mechanical Properties

In this research on flexural-torsional buckling failures elastic-perfect plastic model can be used in the simulation. However, higher accuracy could be obtained with strain hardening models because the measured stress-strain curves showed significant nonlinearities at elevated temperatures. Therefore a strain hardening model was used in the simulations. Appropriate mechanical properties for the selected cold-formed steels were obtained from [12,13]. These values are reported in Table 2. The Poisson's ratio was assumed as 0.3 at ambient and elevated temperatures. Elevated temperature mechanical properties (f_{yT} and E_T) from Table 2 were used with the Ramberg Osgood stress-strain model and appropriate parameters proposed in [12,13] to derive the stress-strain curves at the required elevated temperatures for use in FEA. The Ramberg Osgood stress-strain model cannot be used for the low strength cold-formed steels at lower temperatures because they show a yield pattern in the stress-strain curves. Therefore the elastic-perfect plastic model was used in the FEA of low strength cold-formed steel columns at lower temperatures up to and including 300°C.

3.3. Loading and Boundary Conditions

Accurate boundary condition is the key factor for accurate simulations. The boundary conditions should reflect the end supports used in the tests. Both ends of the test columns were fixed against rotations and translations except that the bottom end was allowed to move axially. These end conditions allowed the specimens to fail symmetrically about the plane perpendicular to the axis of the column at mid-height. Due to the symmetry conditions of the test specimen and loading, it is economical to simulate one half of the test columns in the analyses. Therefore only one half of the column length was simulated with appropriate boundary conditions as shown in Figure 4.

The axial compression load was applied to the test columns by a hydraulic jack via the bottom loading shaft. Therefore axial translation was allowed at the bottom end while keeping the top end fixed. To simulate this, only axial translation at the bottom end was allowed in the finite element model. These boundary conditions were applied to the independent node of the rigid fixed MPC (Multi Point Constraint) located at the bottom end of the model. Rigid fixed MPC consists of an independent node and any number of dependent nodes. Dependent nodes are connected to the independent node using rigid beams with all six structural degrees of freedoms rigidly attached to each other. In this model, the independent node was located at the geometric centre of the cross-section. All perimeter nodes at the bottom surface of the column were considered as the dependent nodes. Since the element size was selected as 5 mm x 5 mm, the distance between the dependent nodes is small. Hence this MPC acts as a rigid surface that is rigidly connected to the bottom end of the columns. In the tests, end plates were rigidly attached to the columns by grouting. Therefore a rigid fixed MPC accurately simulates the end support used for the tests. At the middle of the model, out of plane displacements were allowed while restricting the axial displacement. In addition, axial rotation was allowed at the middle of the model.

3.4. Initial Geometric Imperfections and Residual Stresses

Imperfections of light gauge cold-formed steel members have a considerable influence on their ultimate loads. Nonlinear analyses need initial geometric imperfections to initiate the appropriate buckling deformations. For the nonlinear analyses, a suitable geometric imperfection is introduced to the appropriate buckling mode obtained from bifurcation buckling analyses. Appropriate eigen mode can be selected by comparing the failure mode

from experimental tests. In most cases, the first eigen mode gives the appropriate elastic buckling mode. The measured global geometric imperfections of the specimens reported in Table 3 were used in the models. The maximum amplitude for the critical flexural-torsional buckling mode was used at the column mid-height.

The residual stress models proposed in [17] have higher residual stresses at the rounded corners of lipped channel sections. However, the press-braked sections used in this research had sharp corners, and the corner radii were negligibly small. In this case the corner regions with higher residual stresses can be neglected. A new set of residual stresses for lipped channel sections without rounded corners was proposed in [18]. These values were used in the finite element of model at ambient temperature. The residual stresses reduce with increasing temperatures. Therefore reduced residual stresses based on Equation 1 were used in the analyses as was done in [18]. However, it was found that the effect of residual stress on the ultimate load of the column was less than 1%.

$$\alpha = 1.0181 - 0.00128T \text{ for } 20 \leq T \leq 800 \quad (1)$$

where, α is the residual stress reduction factor and T is the temperature in °C.

3.5. Analyses

As an initial approach, finite strip analyses using CUFSM were conducted to determine the elastic buckling loads and buckling half wave lengths of all the trial sections. The selected specimens were then analysed using ABAQUS to determine the elastic buckling and ultimate loads. MSC/PATRAN [19] was used as a pre-processor to create the input file and as a post-processor to read the results. Two types of analyses, the bifurcation buckling analysis to determine the elastic buckling loads and modes and the nonlinear analysis to determine the ultimate loads and deformations, were undertaken using ABAQUS. Nonlinear analyses were undertaken using the modified Riks method to find the ultimate compression load. In the nonlinear analyses, the maximum load increment was controlled appropriately to obtain smooth loading and accurate load factors.

3.6. Validation

It is important to validate the developed finite element model to determine whether it can simulate the desired buckling and ultimate strength behaviour of cold-formed steel columns. Usually test results are used for validation purposes. In this research, the ultimate failure load,

load-deflection curves and deflected shape from FEA were compared with corresponding test results reported in Bandula Heva and Mahendran [5].

3.6.1. Load-Deflection Curves

Three deflection measurements were recorded in the tests. They are the out-of-plane deflections of web and flange elements and the axial shortening of test specimens. These deflections could also be obtained from finite element simulations. Figures 5 and 6 compare the load-deflection curves from FEA and tests for G450-1.90-2800 Specimens at 20 and 700°C, respectively. Since the load was applied by manual operation of a hydraulic pump, the load-deflection curves from the tests are not smooth as in FEA. Axial shortening of the tests was measured at the bottom end of the loading shaft. Hence the axial shortening in test curves includes the shortening of loading shafts. In addition, it also includes the compaction of insulation packing kept between the end plates and the plates on loading shafts. Therefore the axial shortening curves from tests are slightly higher than the FEA curves. Further, out-of-plane deflections were measured using 800 mm LVDTs, which may involve a maximum error of 2%. Having considered these facts, it can be concluded that the load-deflection curves showed a reasonably good agreement between FEA and tests.

3.6.2. Failure Modes

Since the elevated temperature tests were carried out inside the furnace, visual observation during the test could only be made in the ambient temperature tests. Figures 7 (a) to (d) show the failure modes observed in some tests at ambient and elevated temperatures and compare them with corresponding failure modes from FEA. These figures show that the failure modes from FEA and test are similar, ie. flexural-torsional buckling mode.

3.6.3. Ultimate Loads

The developed finite element model was finally validated by comparing the ultimate loads obtained from tests and FEA for different steel grades, thickness and lengths (Table 3). The mean value of Test/FEA ultimate load ratio is close to one (1.037) while the associated coefficient of variation is also small (0.131). Therefore the developed finite element model can be considered to accurately to simulate the behaviour of long cold-formed steel columns subject to flexural-torsional buckling effects at ambient and elevated temperatures.

4. Design Rules for Cold-formed Steel Columns

4.1. AS/NZS 4600 and AISI S100

The unified approach developed by Pekoz [20] is used in AISI S100 [15] and AS/NZS 4600 [14] for the design of cold-formed steel structural members. According to these two standards, the nominal axial strengths N_c (member capacity) of concentrically loaded compression members are calculated using Equations 2(a) to (c).

$$N_c = A_e f_n \quad (2a)$$

where A_e is the effective area calculated at the critical stresses f_n , which accounts for overall instability, and is determined as,

$$f_n = (0.658^{\lambda_c^2}) f_y \quad \text{for } \lambda_c \leq 1.5 \quad (2b)$$

$$f_n = \left(\frac{0.877}{\lambda_c^2} \right) f_y \quad \text{for } \lambda_c > 1.5 \quad (2c)$$

where λ_c is the non-dimensional slenderness given by,

$$\lambda_c = \sqrt{\frac{f_y}{f_{cre}}} \quad (2d)$$

where f_{cre} is the minimum of elastic buckling stresses f_{ox} , f_{oy} , f_{oz} and f_{oxz} ; f_{ox} and f_{oy} are the elastic flexural buckling stresses about major and minor axes, respectively; f_{oz} and f_{oxz} are the elastic torsional and flexural-torsional buckling stresses, respectively. Further details of the current design rules can be found in AS/NZS 4600 [14] and AISI S100 [15]. Table 3 presents the predicted elastic buckling loads (P_{cr}) and ultimate compression capacities (N_c) of tested columns using the above equations.

Table 3 also presents the ultimate loads of tested columns based on AS/NZS 4600 using gross area (neglecting any local buckling effects), which can be compared with those predicted by AS/NZS 4600 using the effective area at f_n (Equation 2(a)). This comparison shows that the predicted capacities are almost identical, indicating the negligible global-local buckling interactions in the tested columns of this study.

4.2. Direct Strength Method (DSM)

Although the effective width method is still widely used, the Direct Strength Method is becoming more popular as it avoids lengthy calculations. According to DSM [1], the global buckling strength (without any local buckling effects) is given by $A_g f_n$ where f_n can be obtained from Equation 2(b) or 2(c) using elastic buckling loads obtained from analyses such as FEA (see Tables 4 to 6). This shows that the predicted capacities based on DSM will be equal to the AS/NZS 4600 capacities given in Table 3. Hence in the following discussions of this paper, reference will only be made to AS/NZS 4600.

4.3. Eurocode 3 Part 1.3 [16]

Eurocode 3 Part 1.3 is for cold-formed steel structures at ambient temperature. For axial compression members, the design buckling resistance $N_{b,Rd}$ is given by,

$$N_{b,Rd} = \chi A_{eff} f_y / \gamma_{M1} \quad (3a)$$

where A_{eff} is the effective area of the cross-section; f_y is the yield strength; γ_{M1} is the partial factor for resistance of members to instability assessed by member checks, which is taken as 1; χ is the appropriate reduction factor for buckling resistance. The value of χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ should be determined from the relevant buckling curve according to

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1 \quad (3b)$$

$$\text{where } \phi = \frac{1}{2} \left[1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \quad (3c)$$

$$\text{and } \bar{\lambda} = \sqrt{\frac{f_y}{f_{cr}} \frac{A_{eff}}{A}} \quad (3d)$$

α is the imperfection factor based on the relevant buckling curve, and is 0.34 for lipped channel sections (buckling curve “b”); f_{cr} is the elastic flexural-torsional buckling stress; A is the gross area.

4.4. Eurocode 3 Part 1.2 [21]

Eurocode 3 Part 1.2 is specifically developed for hot-rolled steel structures in a fire situation. However, it is also applicable to cold-formed steel members within the scope of Eurocode 3 Part 1.3. The design buckling resistance of Class 1 to 3 compression members at elevated temperatures can be found using Equation 3(a) using appropriately reduced mechanical properties at elevated temperatures. The reduction factor χ for buckling resistance can be calculated using Equation 3(b),

$$\text{where } \phi = \frac{1}{2} \left[1 + \alpha \bar{\lambda}_\theta + \bar{\lambda}_\theta^2 \right] \quad \text{and} \quad \bar{\lambda}_\theta = \sqrt{\frac{f_{y,\theta}}{f_{cr,\theta}}} \frac{A_{eff}}{A} \quad (4a)$$

$f_{y,\theta}$ is the yield stress at elevated temperature θ and $f_{cr,\theta}$ is the elastic flexural-torsional buckling stress at elevated temperature θ .

The imperfection factor α is defined by $\alpha = 0.65 \sqrt{235/f_y}$

4.5. Capacity Reduction Factor

The American cold-formed steel structures code [15] recommends a statistical model to determine a capacity reduction factor to allow for the variations in material, fabrication and the loading effects. This factor is given by Equation 5(a).

$$\phi = 1.52 M_m F_m P_m e^{-\beta_0 \sqrt{V_m^2 + V_f^2 + C_p V_p^2 + V_q^2}} \quad (5a)$$

where, M_m , V_m = Mean and coefficient of variation of the material factor = 1.1, 0.1; F_m , V_f = Mean and coefficient of variation of the fabrication factor = 1, 0.05; V_q = coefficient of variation of load effect = 0.21; β_0 = Target reliability index = 2.5; P_m = mean value of the tested to predicted load ratio; V_p = Coefficient of variation of the tested to predicted load ratio; n = Number of tests; m = Degree of freedom = $n-1$; V_p and P_m values have to be determined from experiments or analyses. In this study ultimate failure loads obtained from FEA were considered. Hence V_p and P_m are the mean and coefficient of variation of the ratio of ultimate failure loads from FEA and design standards; C_p = Correction factor depending on the number

$$\text{of tests} = \left(1 + \frac{1}{n} \right) \left(\frac{m}{m-2} \right)$$

The substitution of all the above values leads to the following equation.

$$\phi = 1.672P_m e^{-2.5\sqrt{0.0566+C_p V_p^2}} \quad (5b)$$

Equation 5(b) was used to determine the capacity reduction factors for the values obtained from DSM, Eurocode 3 Part 1.3 and the proposed design rules. In this study, AS/NZS 4600 [14], AISI [15] and DSM [1] were considered in the design of concentrically loaded compression members. However, the design rules for global buckling (without any local buckling effects) according to DSM are identical to those in AISI S100 and AS/NZS 4600, and hence only AS/NZS 4600 is mentioned in Tables 4 to 6 and 8 to 15.

5. Parametric Study of Lipped Channel Columns

In this parametric study, the validated finite element model described in the last section was used with appropriate changes to the geometric parameters and boundary conditions. Both pinned and fixed ended columns were considered at ambient and uniform elevated temperatures. Figure 4 shows the boundary conditions used in FEA for fixed ended columns. Pinned ended columns were simulated similarly with nodal loads and “1,2,6” boundary conditions at the column end. Instead of the measured thicknesses and imperfections, nominal thicknesses and geometric imperfection values ($L/1000$) were used. Reduced mechanical properties in Table 2 were used while corresponding true stress-strain curves based on Ramberg-Osgood model at elevated temperatures were obtained from [12,13].

5.1. Flexural-torsional Buckling Behaviour of Pinned Ended Columns at Ambient and Elevated Temperatures

One of the major objectives of this research is to assess the suitability of ambient temperature design rules for elevated temperature design of pinned ended columns using reduced mechanical properties. For this purpose, a series of analyses was undertaken for pinned ended columns with G250-0.95, G250-1.95 and G450-1.90 steel sections at ambient and elevated temperatures. The lipped channel section chosen was 55x35x9 mm. Tables 4 to 6 present the ultimate loads (P_{ult}) obtained from FEA, and the ratios of these ultimate loads to the corresponding predictions based on the ambient temperature design rules using the reduced mechanical properties given in Table 2. The buckling curve ‘b’ was used in the calculations based on Eurocode 3 Part 1.3.

The mean values of the ratio of ultimate loads from FEA and AS/NZS 4600 are 1.076, 1.006 and 1.060 with smaller coefficients of variations of 0.025, 0.062 and 0.029, respectively (Tables 4 to 6). Figure 8 (a) shows the graphical representation of the results and AS/NZS 4600 design curve. These comparisons show that the current design rules based on AS/NZS 4600 can be used to predict the ultimate loads of pinned ended cold-formed steel columns subject to flexural-torsional buckling at both ambient and uniform elevated temperatures by using the appropriately reduced mechanical properties of steels at elevated temperatures.

In the case of Eurocode 3 Part 1.3, the mean values of the ratio of ultimate loads from FEA and design code are higher than those obtained for AS/NZS 4600, ie. 1.096, 1.125 and 1.111 (Tables 4 to 6). Figure 8 (b) shows the graphical representation of ultimate load results and different buckling curves of Eurocode 3 Part 1.3, where N_u and N_s are the axial compression member and section capacities, respectively, and $\bar{\lambda}$ is defined by Equation 3(d). This figure shows that the buckling curve “a” is more appropriate to predict the flexural-torsional buckling capacity of pinned ended columns. However, buckling curve “b” provides safer lower bound predictions.

5.2. Flexural-torsional Buckling Behaviour of Fixed Ended Columns

Three different steel grades and thicknesses (G450-1.90 mm, G250-1.95 mm and G550-0.95 mm) were selected in this parametric study to investigate the behaviour of fixed ended cold-formed steel columns at ambient and uniform elevated temperatures subject to flexural-torsional buckling (Table 7). Three lipped channel cross-sections were selected for each thickness to obtain a wide range of results. For each cross-section a range of member lengths (1500 to 4000 mm) was selected to obtain flexural-torsional buckling as the failure mode. Elevated temperatures for this study were selected as 100 °C to 700°C at 100 °C intervals. For some of the shorter columns, the effective area calculated using AS/NZS 4600 effective width equations was slightly less than the gross area, implying the possible occurrence of local and global buckling interaction. Such columns were not included in the comparisons and discussions of this paper.

The ultimate loads of cold-formed steel compression members subject to flexural-torsional buckling at ambient and elevated temperatures were compared with the predictions of ambient and elevated temperature design standards (AISI S100, AS/NZS 4600, DSM and Eurocode 3

Parts 1.2 and 1.3) using the reduced mechanical properties at elevated temperatures. Fixed-end supports provide restraints against major and minor axis rotations as well as twist rotations and warping. Hence fixed ended columns failing by overall buckling were designed as concentrically loaded compression members using their effective length as equal to half the column length. In this case, the effective lengths for major and minor axis flexural buckling as well as torsional buckling were assumed to be equal to one half of the column length.

5.2.1. Ambient Temperature

The ultimate loads at ambient temperature were compared with AS/NZS 4600 and Eurocode 3 Part 1.3 predictions (Table 8). This table also includes a capacity reduction factor in each case, calculated based on the AISI procedure with a reliability index of 2.5 for cold-formed steel members.

Figure 9(a) shows the ultimate loads of fixed ended columns subjected to flexural-torsional buckling, in the form of non-dimensionalised ultimate stress versus slenderness curves. The plots are more scattered for shorter columns (where λ_c is less than or equal to 1.5) and they merge together well with increasing slenderness. Comparative results showed that AS/NZS 4600, AISI S100 and DSM predictions are reasonable for shorter and intermediate columns subject to flexural-torsional buckling. However, for longer columns (second region where $\lambda_c > 1.5$) their predictions (Equation 2(c)) were found to be too conservative.

This behaviour was investigated in detail at ambient temperature by previous researchers [22-24]. Gunalan and Mahendran [24] found that the warping fixity provided by fixed ends enhance the ultimate capacity of cold-formed steel lipped channel columns subject to flexural-torsional buckling. Hence they investigated the second region for slender columns (where λ_c is greater than 1.5) defined by Equation 2(c), and proposed a new set of equations for fixed ended columns with warping fixity when they are subjected to flexural-torsional buckling.

$$f_n = \left(0.69^{\lambda_c^2}\right) f_y \quad \text{for } \lambda_c \leq 1.5 \quad (6a)$$

$$f_n = \left(\frac{0.8}{\lambda_c^{1.5}}\right) f_y \quad \text{for } \lambda_c > 1.5 \quad (6b)$$

The ultimate loads of these columns agreed well with the proposed equations compared to the design curve based on AS/NZS 4600 as shown in Figure 9(a) and Table 8 (mean value is 1.005 and associated coefficient of variation is 0.05).

In the case of Eurocode 3 Part 1.3, it appears to be too conservative as shown in Table 8. The mean value of FEA to code prediction ratio is 1.335 while the associated capacity reduction factor is also much higher than 0.85. Eurocode 3 Part 1.3 recommends the buckling curve “b” for cold-formed steel lipped channel sections. However, comparison of FEA results with the different buckling curves in Eurocode 3 Part 1.3 shows that buckling curve “b” is too conservative (Figure 9(b)). Instead it appears that the ultimate load results follow buckling curve “a₀”. As observed in the case of AS/NZS 4600, the ratio of FEA to Eurocode 3 prediction increases with increasing slenderness (Figure 9(b)) due to the beneficial effect of warping restraint. However, it is not possible to propose any improvements to the design curves within Eurocode 3 Part 1.3 guidelines.

5.2.2. Elevated Temperatures

Tables 9 to 15 compare the FEA results with existing and proposed equations for fixed ended columns subjected to flexural-torsional buckling at different uniform elevated temperatures with varying steel grades, thicknesses, lengths and section sizes. In each case the mean values and associated coefficients of variation (COV) of FEA to code predictions were calculated. The corresponding capacity reduction factors (ϕ) were also calculated.

5.2.2.1. AS/NZS 4600

The mean value and the capacity reduction factor according to AS/NZS 4600 (Equations 2(b) and (c)) are much higher than the target values of 1 and 0.85 respectively, which emphasize the need for improved design rules (Tables 9 to 15). However, the mean value and the capacity reduction factor according to the equations proposed by Gunalan and Mahendran [24] (Equations 6(a) and (b)) are close to the target values.

In the current study it was observed that the ultimate loads of fixed ended columns agreed well with the proposed equations [24] compared to those in AS/NZS 4600 as shown in Figure 10. Figures 11 (a) to (c) show this variation separately for each grade of steel columns. The stress-strain curve of low grade steels has a significant nonlinear region than in high grade steels. Nonlinear behaviour of low strength steels at lower elevated temperatures commences

well before the yielding point. Usually flexural-torsional buckling failures occur in the elastic region. Since low strength steel loses their linear behaviour well below the 0.2% proof stress, failure occurs before it reaches the 0.2% proof stress. Therefore the axial compression load capacity is reduced at 400 and 500°C for low grade steel columns as seen in Figure 11(c). Based on the current study it was found that the ambient temperature design equations proposed by Gunalan and Mahendran [24] can be used to predict the ultimate compression capacities of fixed ended columns reasonably well when they are subjected to flexural torsional buckling, by using the reduced mechanical properties at elevated temperatures. However, further research is needed into the effect of nonlinear stress-strain behaviour on the flexural-torsional buckling behaviour of cold-formed steel columns at elevated temperatures.

5.2.2.2. Eurocode 3 Part 1.3

Eurocode 3 Part 1.3 predictions are conservative than AS/NZS 4600, AISI S100 and DSM design rules. As seen in the comparison for the Australian and American design standards, higher slender columns gave conservative results than lower slender columns. Eurocode 3 Part 1.3 recommends the buckling curve “b” for channel sections. However, comparison of results showed that buckling curve “b” produces more conservative results than buckling curves “a₀” and “a”, particularly for high strength steels (see Figures 12 (a) to (c)). In the case of low strength steels at 400°C and 500°C, results are unconservative even for predictions with buckling curve “b” due to significant nonlinearity in the stress-strain curve before yielding. For all the other cases buckling curve “a” provides conservative predictions while buckling curve “a₀” produce more accurate results. However, some results are between these two buckling curves. Considering the distribution of results, buckling curves given in Table 16 are recommended for fixed ended lipped channel columns at elevated temperatures based on the steel grade and temperatures.

5.2.2.3. Eurocode 3 Part 1.2

Eurocode 3 Part 1.2 is the only available elevated temperature design standard which provides design rules for the fire design of cold-formed steel members. Its predictions for Class 3 sections are overly conservative because imperfection factors based on ambient temperature yield stress are higher than that of buckling curve “b” recommended by Eurocode 3 Part 1.3. Therefore considering the results reported in Tables 9 to 15, Eurocode 3 Part 1.2 is not recommended for the fire design of cold-formed steel compression members since it provides overly conservative capacity predictions.

6. Conclusions

This paper has described a numerical study of the flexural-torsional buckling behaviour of cold-formed steel lipped channel columns at ambient and uniform elevated temperatures. Suitable finite element models were developed and validated using the results obtained from the experimental study conducted by the authors and used further in a parametric study. The ultimate load results were compared with the predictions from the ambient temperature cold-formed steel design standards using appropriately reduced mechanical properties. Elevated temperature capacity predictions of AS/NZS 4600, AISI S100, DSM and Eurocode 3 Part 1.3 using reduced mechanical properties showed a good agreement and a better consistency for pinned ended columns subjected to flexural-torsional buckling. However, they were too conservative for fixed ended slender columns subject to flexural-torsional buckling. This is due to the fact that these design rules ignore the beneficial effect of warping restraint provided by fixed ended columns. Hence it was concluded that the recently proposed equations within AS/NZS 4600 and AISI S100 design provisions should be used with appropriately reduced mechanical properties to accurately predict the flexural-torsional buckling capacity of cold-formed steel compression members at elevated temperatures. The comparative study of ultimate load results from finite element analyses and design codes showed that the capacity of low grade steel columns with intermediate slenderness was adversely affected by the increased nonlinearity in the elevated temperature stress-strain curves. Further research is required to include the effects of nonlinear stress-strain characteristics within the guidelines of AS/NZS 4600 and AISI S100. However, suitable buckling curves can be used for this purpose within Eurocode 3 Part 1.3 design provisions. In contrast, the current Eurocode 3 Part 1.2 design rules were found to be overly conservative.

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Table 1: Details of Test Specimens

Steel Grade	f_y (MPa)	E (MPa)	Nominal thickness (mm)	Measured BMT (mm)	Section dimensions (mm)			Length of specimens (mm)	
					Web	Flange	Lip	Series 1	Series 2
G550	615	205000	0.95	0.95	55	35	9	1800	2800
G250	271	188000	1.95	1.95	75	50	15	1800	2800
G450	515	206000	1.90	1.88	75	50	15	1800	2800

Table 2: Mechanical Properties of Selected Cold-formed Steels

Steel	Temperature (°C)	E_T (MPa)	f_{yT} (MPa)
G550-0.95	20	205000	615
	100	205000	605
	200	178300	596
	300	151600	553
	400	125000	443
	500	98300	276
	600	71700	58
	700	45000	48
G250-0.95	20	200000	320
	100	186640	307
	200	169940	291
	300	143020	210
	400	116020	153
	500	89020	108
	600	62020	71.1
	700	35020	39.8
G450-1.90	20	206000	515
	100	192239	515
	200	175038	510
	300	147311	489
	400	119501	357
	500	91691	201
	600	63881	56.7
	700	36071	36.1
G250-1.95	20	188000	271
	100	175442	260
	200	159744	247
	300	134439	178
	400	109059	129
	500	83679	91.4
	600	58299	60.2
	700	32919	33.7

Table 3: Ultimate Failure Loads from Tests and FEA for Fixed Ended Columns

Test	Steel	T (°C)	Web (mm)	Flange (mm)	Lip (mm)	Length (mm)	Imp. (mm)	AS/NZS 4600			P _{ult} (kN)		Test/FEA	Test/ AS/NZS 4600	AS/NZS 4600* N _c (kN)	Eqs. 6(a) and 6(b) (kN)	Test/ Eqs. 6(a) and 6(b)
								P _{cr} (kN)	N _c (kN)	Test	FEA						
1	G550-0.95	20	54.9	34.9	8.0	1740	0.70	24.7	20.6	24.7	25.4	0.97	1.20	21.7	26.5	0.93	
2		200	54.9	34.8	7.9		0.86	20.9	17.4	22.9	22.5	1.02	1.32	18.3	23.3	0.98	
3		300	55.0	34.8	7.7		0.64	17.4	14.4	21.3	19.3	1.11	1.48	15.3	20.1	1.06	
4		400	55.0	34.5	8.0		0.69	14.3	11.9	20.4	15.8	1.29	1.72	12.5	16.0	1.28	
5		600	54.7	34.8	7.6		0.24	7.5	5.2	6.5	6.2	1.05	1.25	5.3	5.7	1.15	
6		700	55.2	34.8	7.8		0.48	4.3	3.1	3.5	4.6	0.77	1.12	3.2	3.5	1.01	
7		20	54.8	34.8	7.8	2820	1.95	10.3	9.1	15.9	15.4	1.03	1.75	9.1	13.8	1.15	
8		200	54.8	34.8	7.7		1.94	8.7	7.7	12.9	13.7	0.94	1.68	7.7	12.1	1.06	
9		400	54.8	34.6	7.8		2.25	6.0	5.2	9.1	9.6	0.95	1.74	5.2	8.3	1.09	
10		500	55.0	34.7	7.8		3.08	4.6	4.0	6.3	7.1	0.89	1.57	4.0	5.9	1.07	
11		600	54.8	34.8	7.7		1.92	3.2	2.8	4.0	3.4	1.19	1.44	2.8	3.3	1.23	
12		700	54.9	34.9	7.6		1.48	1.8	1.6	2.4	2.3	1.06	1.55	1.6	1.9	1.28	
13	G250-1.95	20	74.8	50.1	14.9	1740	0.68	138.6	76.0	87.9	90.7	0.97	1.16	76.0	78.7	1.12	
14		200	74.8	49.8	15.1		0.43	118.4	67.8	83.8	84.3	0.99	1.24	67.8	70.4	1.19	
15		300	74.8	49.9	15.1		0.46	99.6	51.3	56.1	62.4	0.90	1.09	51.3	53.0	1.06	
16		400	74.9	50.2	14.8		0.58	80.3	38.3	45.4	41.1	1.10	1.19	38.3	39.4	1.15	
17		500	74.9	50.0	15.3		0.48	62.9	27.9	32.2	26.6	1.21	1.16	27.9	28.6	1.13	
18		600	75.4	50.0	15.0		0.32	43.8	18.6	23.0	19.0	1.21	1.24	18.6	19.1	1.21	
19		700	74.8	49.7	14.9	2820	0.38	24.1	10.3	12.6	10.9	1.16	1.22	10.3	10.6	1.19	
20		20	75.0	49.9	14.7		0.69	57.8	48.9	54.1	56.3	0.96	1.11	48.9	53.3	1.01	
21		200	74.8	49.9	15.1		0.74	49.6	42.6	48.8	48.5	1.01	1.15	42.6	46.6	1.05	
22		300	75.0	49.8	14.9		0.55	41.6	34.4	40.9	40.4	1.01	1.19	34.4	37.2	1.10	
23		400	74.9	49.8	14.9		0.66	33.6	26.7	34.7	26.8	1.29	1.30	26.7	28.6	1.21	
24		500	74.7	49.8	14.9		0.58	25.7	19.8	29.9	21.4	1.40	1.51	19.8	21.1	1.42	
25	G450-1.90	600	75.1	49.7	14.6	1740	0.58	17.9	13.4	18.4	15.9	1.16	1.37	13.4	14.3	1.29	
26		700	74.6	49.6	15.0		0.39	10.1	7.6	10.5	9.3	1.13	1.39	7.6	8.0	1.31	
27		20	74.7	49.9	14.5		0.59	143.5	106.0	120.4	129.0	0.93	1.14	109.6	116.8	1.03	
28		200	75.4	49.8	15.1		0.89	126.6	96.3	106.0	110.0	0.96	1.10	101.7	109.2	0.97	
29		300	74.8	49.9	14.8		0.58	104.3	81.6	83.2	96.4	0.86	1.02	87.8	95.4	0.87	
30		400	74.6	49.7	14.7		0.40	83.5	64.6	73.4	78.0	0.94	1.14	68.3	73.6	1.00	
31		500	75.0	49.8	14.8	0.44	65.0	45.0	46.4	53.4	0.87	1.03	46.3	48.9	0.95		
32		600	74.9	49.8	14.9	0.35	45.3	17.3	17.9	20.5	0.87	1.03	17.4	17.8	1.00		
33		700	74.7	49.9	14.8	0.45	25.5	10.7	11.5	11.3	1.01	1.07	10.8	11.1	1.03		
34		20	74.3	49.8	14.9	1.36	60.1	52.7	61.3	63.8	0.96	1.16	52.7	64.3	0.95		
35		200	74.6	49.7	15.1	1.10	51.7	45.4	50.6	56.4	0.90	1.12	45.4	57.4	0.88		
36		400	74.9	49.9	14.8	1.52	35.3	31.0	43.1	38.2	1.13	1.39	31.0	39.4	1.09		
37	500	74.8	49.9	14.7	1.18	26.9	23.6	29.6	27.2	1.09	1.25	23.6	27.8	1.06			
38	600	74.6	49.8	14.9	0.95	18.8	13.1	17.2	15.5	1.11	1.31	13.2	13.9	1.23			
39	700	74.7	50.0	14.8	0.55	10.6	7.9	10.0	9.5	1.05	1.26	7.9	8.4	1.18			
						Mean					1.037	1.285				1.102	
						COV					0.131	0.161				0.113	
						φ						1.039				0.949	

* Ultimate loads based on gross area neglecting any local buckling effects

Table 4: FEA Results for Pinned Ended G250-0.95 Steel Columns

Temp. (°C)	Length (mm)	P_y (kN)	FEA (kN)		Ultimate Load Ratio	
			EBL	P_{ult}	FEA/ AS 4600	FEA/ EC3 P1.3
20	1800	43.5	7.6	7.2	1.08	1.13
	2400	43.5	4.9	4.8	1.10	1.11
	2800	43.5	4.0	3.9	1.10	1.09
100	1800	41.7	7.1	6.7	1.08	1.13
	2400	41.7	4.6	4.5	1.11	1.11
	2800	41.7	3.7	3.6	1.10	1.09
200	1800	39.5	6.5	6.2	1.09	1.13
	2400	39.5	4.2	4.1	1.11	1.11
	2800	39.5	3.4	3.3	1.11	1.09
300	1800	28.5	5.4	4.9	1.02	1.07
	2400	28.5	3.5	3.3	1.06	1.07
	2800	28.5	2.8	2.7	1.07	1.07
400	1800	20.8	4.4	3.9	1.02	1.07
	2400	20.8	2.8	2.7	1.06	1.08
	2800	20.8	2.3	2.2	1.07	1.07
500	1800	14.7	3.4	3.1	1.03	1.10
	2400	14.7	2.2	2.1	1.07	1.09
	2800	14.7	1.8	1.7	1.08	1.08
600	1800	9.7	2.4	2.2	1.04	1.11
	2400	9.7	1.5	1.4	1.08	1.10
	2800	9.7	1.2	1.2	1.09	1.09
700	1800	5.4	1.3	1.2	1.05	1.12
	2400	5.4	0.9	0.8	1.08	1.11
	2800	5.4	0.7	0.7	1.09	1.09
Mean					1.076	1.096
COV					0.025	0.018
ϕ					0.988	1.009

P_y – Yield/Squash Load; EBL – Elastic buckling load from FEA

Table 5: FEA Results for Pinned Ended G250-1.95 Steel Columns

Temp. (°C)	Length (mm)	P _y (kN)	FEA (kN)		Ultimate Load Ratio	
			EBL	P _{ult}	FEA/ AS 4600	FEA/ EC3 P1.3
20	1104	75.6	43.1	38.2	1.05	1.21
	1800	75.6	23.0	21.6	1.07	1.15
100	1104	72.5	40.2	35.8	1.05	1.21
	1800	72.5	21.4	20.2	1.07	1.15
200	1104	68.9	36.6	32.9	1.05	1.21
	1800	68.9	19.5	18.4	1.08	1.14
300	1104	49.6	30.8	22.0	0.87	1.00
	1800	49.6	16.4	14.0	0.97	1.05
400	1104	36.0	25.0	17.6	0.89	1.02
	1800	36.0	13.3	11.4	0.97	1.07
500	1104	25.5	19.2	13.9	0.95	1.08
	1800	25.5	10.2	8.9	1.00	1.11
600	1104	16.8	13.4	9.9	1.00	1.13
	1800	16.8	7.1	6.3	1.01	1.14
700	1104	9.4	7.5	5.8	1.03	1.16
	1800	9.4	4.0	3.6	1.02	1.15
Mean					1.006	1.125
COV					0.062	0.058
ϕ					0.906	1.016

Table 6: FEA Results for Pinned Ended G450-1.90 Steel Columns

Temp. (°C)	Length (mm)	P _y (kN)	FEA (kN)		Ultimate Load Ratio	
			EBL	P _{ult}	FEA/ AS 4600	FEA/ EC3 P1.3
20	1104	139.9	45.4	42.5	1.07	1.17
	1800	139.9	24.0	23.1	1.10	1.12
	2400	139.9	17.6	16.2	1.05	1.04
100	1104	139.9	42.4	40.0	1.08	1.17
	1800	139.9	22.4	21.6	1.10	1.11
	2400	139.9	16.4	15.1	1.05	1.04
200	1104	138.6	38.6	36.4	1.07	1.16
	1800	138.6	20.4	19.7	1.10	1.11
	2400	138.6	15.0	13.8	1.05	1.03
300	1104	132.9	32.5	30.7	1.08	1.15
	1800	132.9	17.1	16.6	1.11	1.11
	2400	132.9	12.6	11.6	1.05	1.03
400	1104	97.0	26.3	24.8	1.07	1.15
	1800	97.0	13.9	13.5	1.10	1.11
	2400	97.0	10.2	9.4	1.05	1.03
500	1104	54.6	20.2	18.6	1.05	1.17
	1800	54.6	10.7	10.2	1.09	1.12
	2400	54.6	7.8	7.2	1.05	1.04
600	1104	15.5	14.1	10.0	1.02	1.14
	1800	15.5	7.4	6.5	1.00	1.15
	2400	15.5	5.5	4.9	1.03	1.12
700	1104	9.8	7.9	6.0	1.02	1.15
	1800	9.8	4.2	3.7	1.02	1.15
	2400	9.8	3.1	2.8	1.04	1.12
Mean					1.060	1.111
COV					0.029	0.044
ϕ					0.972	1.012

Table 7: Parameters Considered in FEA

Index	Grade	Thickness (mm)	Web (mm)	Flange (mm)	Lip (mm)	Length (mm)
G250-1.95-60-1500	G250	1.95	60	40	15	1500
G250-1.95-60-2000						2000
G250-1.95-60-2500						2500
G250-1.95-60-3000						3000
G250-1.95-60-3500						3500
G250-1.95-75-1500			75	50	15	1500
G250-1.95-75-2000						2000
G250-1.95-75-2500						2500
G250-1.95-75-3000						3000
G250-1.95-75-3500						3500
G250-1.95-90-2500		1.95	90	60	15	2500
G250-1.95-90-3000						3000
G250-1.95-90-3500						3500
G250-1.95-90-4000						4000
G450-1.90-60-1500	G450	1.90	60	40	15	1500
G450-1.90-60-2000						2000
G450-1.90-60-2500						2500
G450-1.90-60-3000						3000
G450-1.90-60-3500						3500
G450-1.90-75-1500			75	50	15	1500
G450-1.90-75-2000						2000
G450-1.90-75-2500						2500
G450-1.90-75-3000						3000
G450-1.90-75-3500						3500
G450-1.90-90-2500			90	60	15	2500
G450-1.90-90-3000						3000
G450-1.90-90-3500						3500
G450-1.90-90-4000						4000
G550-0.95-40-1500	G550	0.95	40	25	8	1500
G550-0.95-40-2000						2000
G550-0.95-40-2500						2500
G550-0.95-55-2500			55	35	8	2500
G550-0.95-55-3000						3000

Table 8: FEA Results for Fixed Ended Columns at 20°C

Index	P _y (kN)	FEA (kN)		Ultimate Load Ratio			
		EBL	P _{ult}	FEA/ AS 4600	FEA/ Eqs. 6(a) and 6(b)	FEA/ EC3 P1.3	FEA/ EC3 P1.2
G250-1.95-60-1500	89.8	125.8	75.0	1.13	1.09	1.19	1.52
G250-1.95-60-2000	89.8	75.4	60.1	1.10	1.04	1.23	1.56
G250-1.95-60-2500	89.8	51.6	46.2	1.07	0.98	1.23	1.51
G250-1.95-60-3000	89.8	38.6	39.9	1.18	1.05	1.33	1.50
G250-1.95-60-3500	89.8	30.9	30.7	1.13	0.95	1.24	1.46
G250-1.95-75-2000	108.3	117.1	82.6	1.12	1.07	1.25	1.58
G250-1.95-75-2500	108.3	78.6	65.8	1.08	1.01	1.25	1.56
G250-1.95-75-3000	108.3	57.2	52.3	1.07	0.97	1.24	1.51
G250-1.95-75-3500	108.3	44.2	43.1	1.11	0.97	1.25	1.50
G250-1.95-90-3000	126.8	82.7	71.6	1.07	1.00	1.27	1.56
G250-1.95-90-3500	126.8	62.7	58.6	1.08	0.98	1.26	1.52
G250-1.95-90-4000	126.8	49.8	49.6	1.14	0.99	1.28	1.52
G450-1.90-60-1500	166.3	133.8	109.0	1.10	1.04	1.27	0.96
G450-1.90-60-2000	166.3	80.1	74.3	1.07	0.97	1.24	1.01
G450-1.90-60-2500	166.3	54.7	55.4	1.15	0.96	1.26	1.07
G450-1.90-60-3000	166.3	40.7	44.8	1.26	0.97	1.32	1.14
G450-1.90-60-3500	166.3	32.2	37.8	1.34	0.97	1.37	1.21
G450-1.90-75-2500	200.6	83.4	79.9	1.09	0.96	1.27	1.02
G450-1.90-75-3000	200.6	60.6	63.8	1.20	0.98	1.32	1.10
G450-1.90-75-3500	200.6	46.7	54.4	1.33	1.01	1.40	1.20
G450-1.90-90-3500	234.8	67.1	72.3	1.23	0.98	1.37	1.12
G450-1.90-90-4000	234.8	53.2	63.3	1.36	1.03	1.46	1.22
G550-0.95-40-1500	61.3	16.1	17.8	1.26	0.99	1.33	1.42
G550-0.95-40-2000	61.3	10.0	13.2	1.51	1.05	1.51	1.59
G550-0.95-40-2500	61.3	7.2	10.2	1.62	1.04	1.60	1.66
G550-0.95-55-2500	81.8	13.2	17.9	1.55	1.07	1.62	1.66
G550-0.95-55-3000	81.8	9.6	14.8	1.76	1.13	1.77	1.80
Mean				1.225	1.009	1.338	1.388
COV				0.152	0.046	0.105	0.171
ϕ				0.998	0.920	1.160	1.098

Table 9: FEA Results for Fixed Ended Columns at 100°C

Index	P _y (kN)	FEA (kN)		Ultimate Load Ratio			
		EBL	P _{ult}	FEA/ AS 4600	FEA/ Eqs. 6(a) and 6(b)	FEA/ EC3 P1.3	FEA/ EC3 P1.2
G250-1.95-60-1500	86.2	117.5	71.8	1.13	1.09	1.20	1.53
G250-1.95-60-2000	86.2	70.4	56.9	1.10	1.04	1.24	1.55
G250-1.95-60-2500	86.2	48.2	43.5	1.07	0.98	1.23	1.49
G250-1.95-60-3000	86.2	36.0	34.7	1.10	0.97	1.23	1.46
G250-1.95-60-3500	86.2	28.5	28.9	1.16	0.96	1.25	1.44
G250-1.95-75-2000	103.9	109.3	78.2	1.12	1.07	1.25	1.58
G250-1.95-75-2500	103.9	73.3	62.2	1.08	1.01	1.25	1.56
G250-1.95-75-3000	103.9	53.4	49.2	1.07	0.97	1.24	1.51
G250-1.95-75-3500	103.9	41.2	40.6	1.12	0.98	1.26	1.50
G250-1.95-90-3500	121.7	59.0	55.1	1.07	0.97	1.27	1.52
G250-1.95-90-4000	121.7	46.8	46.7	1.14	0.98	1.29	1.52
G450-1.90-60-1500	166.3	124.9	104.0	1.09	1.02	1.27	1.41
G450-1.90-60-2000	166.3	74.7	70.3	1.07	0.97	1.24	1.35
G450-1.90-60-2500	166.3	51.0	52.7	1.18	0.96	1.28	1.37
G450-1.90-60-3000	166.3	38.0	42.7	1.28	0.97	1.34	1.42
G450-1.90-60-3500	166.3	30.1	36.0	1.36	0.98	1.39	1.46
G450-1.90-75-2500	200.6	77.9	75.8	1.11	0.96	1.28	1.37
G450-1.90-75-3000	200.6	56.6	60.8	1.22	0.98	1.33	1.41
G450-1.90-75-3500	200.6	43.6	51.8	1.35	1.01	1.43	1.50
G450-1.90-90-3500	234.8	62.3	69.2	1.27	1.00	1.39	1.46
G450-1.90-90-4000	234.8	49.6	60.8	1.40	1.04	1.49	1.55
G550-0.95-40-1500	60.3	16.1	17.6	1.25	0.98	1.32	1.41
G550-0.95-40-2000	60.3	10.0	13.1	1.49	1.04	1.50	1.58
G550-0.95-40-2500	60.3	7.2	10.1	1.60	1.03	1.58	1.65
G550-0.95-55-2500	80.5	13.2	17.9	1.55	1.08	1.63	1.66
G550-0.95-55-3000	80.5	9.6	14.8	1.76	1.13	1.77	1.81
Mean				1.237	1.007	1.344	1.503
COV				0.153	0.046	0.107	0.068
φ				1.007	0.917	1.163	1.349

Table 10: FEA Results for Fixed Ended Columns at 200°C

Index	P _y (kN)	FEA (kN)		Ultimate Load Ratio			
		EBL	P _{ult}	FEA/ AS 4600	FEA/ Eqs. 6(a) and 6(b)	FEA/ EC3 P1.3	FEA/ EC3 P1.2
G250-1.95-60-1500	81.9	107.0	67.7	1.14	1.10	1.21	1.55
G250-1.95-60-2000	81.9	64.1	52.9	1.10	1.04	1.24	1.57
G250-1.95-60-2500	81.9	43.9	40.1	1.07	0.98	1.23	1.51
G250-1.95-60-3000	81.9	32.8	32.0	1.11	0.97	1.24	1.48
G250-1.95-60-3500	81.9	26.0	26.7	1.17	0.96	1.26	1.47
G250-1.95-75-2000	98.7	99.6	73.5	1.13	1.08	1.27	1.59
G250-1.95-75-2500	98.7	60.8	57.5	1.15	1.06	1.26	1.56
G250-1.95-75-3000	98.7	48.6	45.4	1.08	0.98	1.25	1.51
G250-1.95-75-3500	98.7	37.6	37.5	1.14	0.98	1.27	1.50
G250-1.95-90-3500	115.6	53.8	50.9	1.08	0.98	1.28	1.52
G250-1.95-90-4000	115.6	42.7	43.1	1.15	0.98	1.30	1.52
G450-1.90-60-1500	164.7	113.7	97.0	1.08	1.01	1.27	1.40
G450-1.90-60-2000	164.7	68.1	64.9	1.09	0.96	1.24	1.34
G450-1.90-60-2500	164.7	46.5	48.8	1.20	0.96	1.29	1.37
G450-1.90-60-3000	164.7	34.6	39.7	1.31	0.97	1.36	1.43
G450-1.90-60-3500	164.7	27.4	33.5	1.39	0.98	1.41	1.48
G450-1.90-75-2500	198.6	70.9	70.1	1.13	0.96	1.28	1.36
G450-1.90-75-3000	198.6	51.5	56.7	1.26	0.98	1.36	1.43
G450-1.90-75-3500	198.6	39.7	48.4	1.39	1.02	1.45	1.52
G450-1.90-90-3500	232.6	57.1	64.7	1.29	1.00	1.42	1.47
G450-1.90-90-4000	232.6	45.2	57.0	1.44	1.05	1.53	1.58
G550-0.95-40-1500	59.5	14.0	15.9	1.29	0.99	1.36	1.43
G550-0.95-40-2000	59.5	8.8	11.9	1.54	1.05	1.56	1.63
G550-0.95-40-2500	59.5	6.3	9.1	1.65	1.03	1.63	1.68
G550-0.95-55-2500	79.3	11.5	15.8	1.57	1.06	1.64	1.66
G550-0.95-55-3000	79.3	8.4	13.0	1.76	1.10	1.78	1.80
Mean				1.258	1.008	1.360	1.514
COV				0.156	0.045	0.111	0.069
ϕ				1.019	0.919	1.171	1.358

Table 11: FEA Results for Fixed Ended Columns at 300°C

Index	P_y (kN)	FEA (kN)		Ultimate Load Ratio			
		EBL	P_{ult}	FEA/ AS 4600	FEA/ Eqs. 6(a) and 6(b)	FEA/ EC3 P1.3	FEA/ EC3 P1.2
G250-1.95-60-1500	59.0	90.0	50.7	1.13	1.10	1.19	1.52
G250-1.95-60-2000	59.0	54.0	41.7	1.12	1.06	1.24	1.58
G250-1.95-60-2500	59.0	36.9	32.3	1.07	0.99	1.23	1.52
G250-1.95-60-3000	59.0	27.6	27.9	1.16	1.05	1.32	1.60
G250-1.95-60-3500	59.0	21.9	21.6	1.12	0.96	1.24	1.47
G250-1.95-75-1500	71.2	139.8	63.8	1.11	1.08	1.17	1.46
G250-1.95-75-2000	71.2	83.8	56.4	1.13	1.09	1.23	1.58
G250-1.95-75-2500	71.2	56.2	45.6	1.09	1.03	1.24	1.56
G250-1.95-75-3000	71.2	40.9	36.7	1.07	0.98	1.24	1.53
G250-1.95-75-3500	71.2	31.6	30.2	1.09	0.98	1.24	1.50
G250-1.95-90-3000	83.3	59.5	50.1	1.08	1.01	1.27	1.58
G250-1.95-90-3500	83.3	45.2	41.1	1.07	0.98	1.26	1.54
G250-1.95-90-4000	83.3	35.9	34.7	1.10	0.98	1.27	1.52
G450-1.90-60-2000	157.9	57.2	55.9	1.11	0.95	1.25	1.33
G450-1.90-60-2500	157.9	39.1	42.4	1.24	0.96	1.31	1.38
G450-1.90-60-3000	157.9	29.1	34.7	1.36	0.98	1.39	1.46
G450-1.90-60-3500	157.9	23.0	29.1	1.44	0.98	1.45	1.51
G450-1.90-75-2500	190.5	59.7	60.6	1.16	0.95	1.30	1.36
G450-1.90-75-3000	190.5	43.4	49.6	1.30	0.99	1.40	1.45
G450-1.90-75-3500	190.5	33.4	42.6	1.45	1.03	1.51	1.56
G450-1.90-90-3500	223.0	48.0	57.0	1.35	1.01	1.47	1.51
G450-1.90-90-4000	223.0	38.0	50.3	1.51	1.06	1.59	1.62
G550-0.95-40-1500	55.2	11.9	13.7	1.31	0.98	1.37	1.43
G550-0.95-40-2000	55.2	7.4	10.2	1.57	1.04	1.56	1.62
G550-0.95-40-2500	55.2	5.3	7.8	1.68	1.02	1.64	1.68
G550-0.95-55-2500	73.5	9.7	13.6	1.60	1.06	1.65	1.66
G550-0.95-55-3000	73.5	7.1	11.2	1.80	1.10	1.79	1.81
Mean				1.268	1.014	1.363	1.531
COV				0.169	0.046	0.121	0.067
ϕ				1.006	0.924	1.160	1.376

Table 12: FEA Results for Fixed Ended Columns at 400°C

Index	P_y (kN)	FEA (kN)		Ultimate Load Ratio			
		EBL	P_{ult}	FEA/ AS 4600	FEA/ Eqs. 6(a) and 6(b)	FEA/ EC3 P1.3	FEA/ EC3 P1.2
G250-1.95-60-1500	42.9	73.0	29.3	0.87	0.85	0.91	1.16
G250-1.95-60-2000	42.9	43.8	24.8	0.87	0.83	0.95	1.22
G250-1.95-60-2500	42.9	30.0	21.1	0.89	0.84	1.02	1.28
G250-1.95-60-3000	42.9	22.4	17.8	0.92	0.84	1.06	1.30
G250-1.95-60-3500	42.9	17.7	15.2	0.98	0.86	1.09	1.31
G250-1.95-75-1500	51.7	113.4	37.8	0.88	0.87	0.91	1.16
G250-1.95-75-2000	51.7	68.0	32.5	0.86	0.83	0.92	1.19
G250-1.95-75-2500	51.7	45.6	28.2	0.88	0.83	0.98	1.25
G250-1.95-75-3000	51.7	33.2	24.2	0.90	0.83	1.03	1.29
G250-1.95-75-3500	51.7	25.6	20.8	0.94	0.85	1.08	1.32
G250-1.95-90-3000	60.6	48.2	31.5	0.88	0.83	1.02	1.29
G250-1.95-90-3500	60.6	36.7	27.4	0.90	0.83	1.06	1.32
G250-1.95-90-4000	60.6	29.1	23.8	0.94	0.85	1.09	1.33
G450-1.90-60-1500	115.3	77.7	65.4	1.06	0.98	1.25	1.37
G450-1.90-60-2000	115.3	46.4	44.0	1.08	0.94	1.23	1.32
G450-1.90-60-2500	115.3	31.7	33.2	1.19	0.95	1.28	1.36
G450-1.90-60-3000	115.3	23.6	27.0	1.30	0.96	1.35	1.42
G450-1.90-60-3500	115.3	18.7	22.8	1.39	0.97	1.41	1.47
G450-1.90-75-2500	139.1	48.4	47.5	1.12	0.94	1.27	1.34
G450-1.90-75-3000	139.1	35.2	38.6	1.25	0.97	1.35	1.42
G450-1.90-75-3500	139.1	27.1	33.0	1.39	1.01	1.45	1.51
G450-1.90-90-3500	162.8	38.9	44.1	1.29	0.99	1.41	1.46
G450-1.90-90-4000	162.8	30.9	38.9	1.44	1.04	1.53	1.57
G550-0.95-40-1500	44.2	9.8	11.2	1.30	0.98	1.36	1.43
G550-0.95-40-2000	44.2	6.1	8.4	1.57	1.05	1.56	1.62
G550-0.95-40-2500	44.2	4.4	6.4	1.66	1.02	1.63	1.68
G550-0.95-55-2500	58.9	8.0	11.1	1.58	1.05	1.63	1.65
G550-0.95-55-3000	58.9	5.9	9.3	1.80	1.11	1.81	1.83
Mean				1.148	0.926	1.238	1.388
COV				0.245	0.094	0.203	0.118
ϕ				0.796	0.813	0.930	1.186

Table 13: FEA Results for Fixed Ended Columns at 500°C

Index	P_y (kN)	FEA (kN)		Ultimate Load Ratio			
		EBL	P_{ult}	FEA/ AS 4600	FEA/ Eqs. 6(a) and 6(b)	FEA/ EC3 P1.3	FEA/ EC3 P1.2
G250-1.95-60-1500	30.3	56.0	22.4	0.93	0.90	0.96	1.22
G250-1.95-60-2000	30.3	33.6	19.5	0.94	0.90	1.02	1.30
G250-1.95-60-2500	30.3	23.0	16.0	0.92	0.86	1.04	1.31
G250-1.95-60-3000	30.3	17.7	14.0	0.95	0.87	1.11	1.37
G250-1.95-60-3500	30.3	13.6	11.9	1.00	0.90	1.13	1.37
G250-1.95-75-1500	36.5	87.0	28.5	0.93	0.91	0.95	1.21
G250-1.95-75-2000	36.5	52.1	25.3	0.93	0.90	0.98	1.27
G250-1.95-75-2500	36.5	35.0	22.2	0.94	0.90	1.04	1.34
G250-1.95-75-3000	36.5	25.5	19.1	0.95	0.89	1.09	1.37
G250-1.95-75-3500	36.5	19.7	16.3	0.97	0.89	1.12	1.38
G250-1.95-90-3000	42.8	37.0	24.8	0.94	0.89	1.08	1.37
G250-1.95-90-3500	42.8	28.5	21.6	0.95	0.88	1.11	1.40
G250-1.95-90-4000	42.8	22.3	18.6	0.97	0.89	1.13	1.39
G450-1.90-60-1500	64.9	59.6	43.7	1.06	1.01	1.20	1.36
G450-1.90-60-2000	64.9	35.6	31.4	1.04	0.95	1.21	1.34
G450-1.90-60-2500	64.9	24.3	23.5	1.10	0.95	1.22	1.33
G450-1.90-60-3000	64.9	18.1	18.8	1.18	0.94	1.26	1.35
G450-1.90-60-3500	64.9	14.3	15.8	1.26	0.95	1.30	1.38
G450-1.90-75-2500	78.3	37.1	33.9	1.05	0.95	1.24	1.35
G450-1.90-75-3000	78.3	27.0	26.8	1.13	0.95	1.26	1.36
G450-1.90-75-3500	78.3	20.8	22.5	1.23	0.97	1.32	1.41
G450-1.90-90-3500	91.7	29.9	30.2	1.15	0.95	1.30	1.39
G450-1.90-90-4000	91.7	23.7	26.2	1.26	0.99	1.37	1.45
G550-0.95-40-1500	27.5	7.7	8.1	1.20	0.96	1.27	1.37
G550-0.95-40-2000	27.5	4.8	6.0	1.43	1.01	1.44	1.53
G550-0.95-40-2500	27.5	3.4	4.7	1.58	1.02	1.54	1.61
G550-0.95-55-2500	36.7	6.3	8.2	1.48	1.05	1.56	1.61
G550-0.95-55-3000	36.7	4.6	6.9	1.71	1.12	1.73	1.77
Mean				1.113	0.940	1.213	1.390
COV				0.194	0.064	0.156	0.084
ϕ				0.849	0.847	0.984	1.231

Table 14: FEA Results for Fixed Ended Columns at 600°C

Index	P_y (kN)	FEA (kN)		Ultimate Load Ratio			
		EBL	P_{ult}	FEA/ AS 4600	FEA/ Eqs. 6(a) and 6(b)	FEA/ EC3 P1.3	FEA/ EC3 P1.2
G250-1.95-60-1500	19.9	39.0	16.3	1.01	0.99	1.05	1.33
G250-1.95-60-2000	19.9	23.4	14.2	1.02	0.98	1.10	1.41
G250-1.95-60-2500	19.9	16.0	12.0	1.02	0.96	1.14	1.44
G250-1.95-60-3000	19.9	12.0	10.0	1.01	0.93	1.16	1.44
G250-1.95-60-3500	19.9	9.5	8.5	1.03	0.93	1.18	1.42
G250-1.95-75-1500	24.0	60.6	19.9	0.98	0.96	1.00	1.27
G250-1.95-75-2000	24.0	36.3	18.3	1.01	0.97	1.06	1.37
G250-1.95-75-2500	24.0	24.4	16.2	1.02	0.97	1.12	1.45
G250-1.95-75-3000	24.0	17.7	13.8	1.01	0.95	1.15	1.46
G250-1.95-75-3500	24.0	13.7	11.7	1.02	0.93	1.17	1.45
G250-1.95-90-2500	28.1	35.4	20.4	1.01	0.98	1.11	1.43
G250-1.95-90-3000	28.1	25.8	18.1	1.02	0.97	1.15	1.48
G250-1.95-90-3500	28.1	19.6	15.6	1.01	0.95	1.17	1.48
G250-1.95-90-4000	28.1	15.6	13.4	1.01	0.93	1.18	1.47
G450-1.90-60-1500	18.4	41.5	16.4	1.07	1.05	1.11	1.30
G450-1.90-60-2000	18.4	24.8	14.4	1.07	1.03	1.14	1.34
G450-1.90-60-2500	18.4	17.0	12.3	1.05	1.00	1.17	1.36
G450-1.90-60-3000	18.4	12.6	10.3	1.03	0.96	1.18	1.35
G450-1.90-60-3500	18.4	10.0	8.8	1.03	0.95	1.19	1.33
G450-1.90-75-1500	22.2	74.8	20.5	1.05	1.03	1.08	1.34
G450-1.90-75-2000	22.2	38.6	18.6	1.07	1.04	1.11	1.37
G450-1.90-75-2500	22.2	25.9	16.5	1.06	1.02	1.15	1.40
G450-1.90-75-3000	22.2	14.5	13.2	1.13	1.05	1.09	1.30
G450-1.90-75-3500	22.2	14.5	12.1	1.03	0.96	1.19	1.38
G450-1.90-90-2500	26.0	38.0	20.8	1.07	1.03	1.14	1.48
G450-1.90-90-3000	26.0	27.4	18.5	1.06	1.01	1.17	1.48
G450-1.90-90-3500	26.0	20.8	16.0	1.04	0.98	1.18	1.45
G450-1.90-90-4000	26.0	16.5	13.8	1.03	0.95	1.19	1.42
G550-0.95-40-1500	5.8	5.6	3.8	1.01	0.96	1.09	1.37
G550-0.95-40-2000	5.8	3.5	2.9	1.00	0.93	1.13	1.34
G550-0.95-40-2500	5.8	2.5	2.3	1.05	0.93	1.17	1.33
G550-0.95-55-2500	7.7	4.6	3.9	1.02	0.94	1.19	1.45
G550-0.95-55-3000	7.7	3.4	3.1	1.04	0.93	1.19	1.41
Mean				1.033	0.974	1.139	1.397
COV				0.027	0.040	0.041	0.044
ϕ				0.948	0.891	1.040	1.275

Table 15: FEA Results for Fixed Ended Columns at 700°C

Index	P_y (kN)	FEA (kN)		Ultimate Load Ratio			
		EBL	P_{ult}	FEA/ AS 4600	FEA/ Eqs. 6(a) and 6(b)	FEA/ EC3 P1.3	FEA/ EC3 P1.2
G250-1.95-60-1500	11.3	22.0	9.1	1.00	0.98	1.04	1.31
G250-1.95-60-2000	11.3	13.2	8.1	1.03	0.99	1.11	1.42
G250-1.95-60-2500	11.3	9.0	6.9	1.03	0.97	1.16	1.46
G250-1.95-60-3000	11.3	6.8	5.7	1.01	0.94	1.17	1.44
G250-1.95-60-3500	11.3	5.4	4.8	1.02	0.92	1.18	1.42
G250-1.95-75-1500	13.6	34.2	11.4	0.99	0.97	1.01	1.28
G250-1.95-75-2000	13.6	20.5	10.4	1.01	0.98	1.06	1.38
G250-1.95-75-2500	13.6	13.8	9.3	1.03	0.99	1.13	1.46
G250-1.95-75-3000	13.6	10.0	7.9	1.03	0.96	1.16	1.47
G250-1.95-75-3500	13.6	7.7	6.6	1.02	0.93	1.17	1.45
G250-1.95-90-2500	15.9	19.0	11.7	1.04	1.00	1.12	1.45
G250-1.95-90-3000	15.9	14.6	10.3	1.02	0.97	1.16	1.49
G250-1.95-90-3500	15.9	11.1	8.9	1.02	0.95	1.18	1.49
G250-1.95-90-4000	15.9	8.8	7.6	1.02	0.93	1.19	1.48
G450-1.90-60-1500	11.6	23.4	9.6	1.02	0.99	1.05	1.24
G450-1.90-60-2000	11.6	14.0	8.5	1.03	0.99	1.11	1.31
G450-1.90-60-2500	11.6	9.9	7.2	1.01	0.96	1.16	1.34
G450-1.90-60-3000	11.6	7.1	6.0	1.02	0.95	1.17	1.33
G450-1.90-60-3500	11.6	5.6	5.0	1.03	0.93	1.17	1.31
G450-1.90-75-1500	14.0	36.4	11.9	1.00	0.98	1.02	1.25
G450-1.90-75-2000	14.0	21.8	11.0	1.03	1.00	1.08	1.33
G450-1.90-75-2500	14.0	14.6	9.7	1.03	0.99	1.13	1.37
G450-1.90-75-3000	14.0	10.6	8.3	1.03	0.97	1.16	1.37
G450-1.90-75-3500	14.0	8.2	7.0	1.02	0.94	1.18	1.35
G450-1.90-90-2500	16.4	21.5	12.3	1.03	0.99	1.13	1.43
G450-1.90-90-3000	16.4	15.5	10.9	1.03	0.98	1.17	1.45
G450-1.90-90-3500	16.4	11.8	9.3	1.01	0.95	1.18	1.42
G450-1.90-90-4000	16.4	9.3	8.0	1.02	0.94	1.19	1.32
G550-0.95-40-1500	4.8	3.5	2.8	1.04	0.97	1.15	1.38
G550-0.95-40-2000	4.8	2.2	2.0	1.04	0.94	1.16	1.34
G550-0.95-40-2500	4.8	1.6	1.6	1.14	0.95	1.24	1.35
G550-0.95-55-2500	6.4	2.9	2.7	1.06	0.96	1.24	1.42
G550-0.95-55-3000	6.4	2.1	2.2	1.19	0.99	1.30	1.43
Mean				1.032	0.965	1.146	1.386
COV				0.037	0.024	0.054	0.051
ϕ				0.945	0.888	1.040	1.260

Table 16: Proposed Buckling Curves for Fire Design Using Eurocode 3 Part 1.3

Grade	Low Grade Steel			High Grade Steel	
Temperature °C	$20^{\circ}\text{C} \leq T \leq 300^{\circ}\text{C}$	$300^{\circ}\text{C} < T \leq 500^{\circ}\text{C}$	$500^{\circ}\text{C} < T \leq 700^{\circ}\text{C}$	$20^{\circ}\text{C} \leq T \leq 500^{\circ}\text{C}$	$500^{\circ}\text{C} < T \leq 700^{\circ}\text{C}$
Buckling Curve	a_0	c	b	a_0	a

Figure 1: Cold-formed Steel Columns Used in Light Gauge Steel Frames

Figure 2: Typical Buckling Plot of Lipped Channel Columns

(a) Series 1 – 1800 mm long columns

(b) Series 2 – 2800 mm long columns

Figure 3: Experimental Study

Figure 4: Loading and Boundary Conditions Used in FEA

(a) Axial compression load versus axial shortening

(b) Axial compression load versus out-of-plane deflection

Figure 5: Comparison of Load-Deflection Curves from Tests and FEA for G450-1.90-2800-20 Specimens

(a) Axial compression load versus axial shortening

(b) Axial compression load versus out-of-plane deflection

Figure 6: Comparison of Load-Deflection Curves from Tests and FEA for G450-1.90-2800-700 Specimens

Figure 7: Comparison of Failure Modes from Tests and FEA

(a) AS/NZS 4600

(b) Eurocode 3 Part 1.3

Figure 8: Comparison of FEA Results with Design Rules for Pinned Ended Columns at Ambient and Elevated Temperatures

(a) AS/NZS 4600

(b) Eurocode 3 Part 1.3

Figure 9: Comparison of FEA Results with Design Rules for Fixed Ended Columns at Ambient Temperature

Figure 10: Comparison of FEA Results with AS/NZS 4600 Design Rules for Fixed Ended Columns at Elevated Temperatures

Figure 11: Comparison of FEA Results with AS/NZS 4600 Design Rules for Fixed Ended Columns with Different Steel Grades

Figure 12: Comparison of FEA Results with Eurocode 3 Part 1.3 Design Rules for Fixed Ended Columns with Different Steel Grades