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NUMERICAL STUDY OF COLD-FORMED STEEL BEAMS SUBJECT TO LATERAL-TORSIONAL BUCKLING

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ABSTRACT

The use of cold-formed steel members as structural columns and beams in residential, industrial and commercial buildings has increased significantly in recent times. This study is focused on the use of cold-formed steel sections as flexural members subject to lateral-torsional buckling. For this purpose a finite element model of a simply supported lipped channel beam under uniform bending was developed, validated using available numerical and experimental results, and used in a detailed parametric study. The moment capacity results were then compared with the predictions from the current ambient temperature design rules in the cold-formed steel structures codes of Australia, New Zealand, North America and Europe. European design rules were found to be conservative while Australian and American design rules were unsafe. This paper presents the results of the numerical study, the comparison with the current design rules and the new proposed design rules.

1. INTRODUCTION

In recent times cold-formed steel structures are increasingly used in buildings. However, Engineers face problems in using cold-formed steel members because of lack of suitable design specifications for fire conditions. Eurocode 3 Part 1.2 [1] and BS 5950 Part 8 [2] provide design rules for hot-rolled steel members at elevated temperatures. However, only Eurocode 3 Part 1.2 [1] allows the use of design rules developed based on hot-rolled steel members for cold-formed steel members at elevated temperatures despite the differences between them. Eurocode 3 Part 1.3 [3], BS5950 Part 5 [4] and AS/NZS 4600 [5] provide design rules for cold-formed steel members at ambient temperature. These design rules may also be used for cold–formed steel members at elevated temperatures by using the reduced mechanical properties at elevated temperatures. The overall aim of this research is to verify the applicability of current ambient temperature design rules for lateral-torsional buckling of cold-formed steel lipped channel beams for elevated temperature conditions. Hence as the first step, the accuracy of ambient temperature design rules was investigated for cold-formed steel flexural members subject to lateral-torsional buckling. Lateral-torsional buckling behaviour of lipped channel cold-formed steel beams at ambient temperature was investigated using a validated finite element model in a detailed parametric study. This paper presents the results and compares them with the current design rules at ambient temperature. Finally the new design rule developed in this research is presented.

2. LATERAL-TORSIONAL BUCKLING DESIGN RULES

2.1. AS/NZS 4600 [5]

The nominal member moment capacity *(M_b)* of beams subject to lateral buckling is given by

$$M_b = Z_c \left(M_c / Z_f \right) \tag{1}$$

Z_c and Z_f are the effective section modulus calculated at a stress level (M_c/Z_f) in the extreme compression fibre, and the full unreduced section modulus for the extreme compression fibre, respectively. The critical moment (M_c) can be calculated as follows,

$$\left.\begin{array}{lll} \text{For} & \lambda_b \leq 0.6 & M_c = M_y \\ \text{For} & 0.60 < \lambda_b < 1.336 & M_c = 1.11 M_y \left\{ 1 - \left(10/36 \right) \lambda_b^2 \right\} \\ \text{For} & \lambda_b \geq 1.336 & M_c = M_y / \lambda_b^2 \end{array}\right\} \tag{2}$$

where, $\lambda_b = \sqrt{M_y / M_o}$ is the non-dimensional slenderness ratio. $M_y = Z_f f_y$ is the first yield moment of the full section and M_o is the elastic lateral-torsional buckling moment.

2.2. Eurocode 3 Part 1.3 [3]

Eurocode 3 Part 1.3 recommends the following design equations based on buckling curve 'b.

$$M_{b,Rd} = \chi_{LT} W_y \, f_y / \gamma_{M1} \tag{3}$$

where $\chi_{LT} = 1 \Big/ \left\{ \phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2 \right]^{0.5} \right\}$, $\phi_{LT} = 0.5 \left[1 + \alpha_{LT} \left(\bar{\lambda}_{LT} - 0.2 \right) + \bar{\lambda}_{LT}^{\,2} \right]$ and $\bar{\lambda}_{LT} = \left[f_y W_y / M_o \right]^{0.5}$

W_y is the appropriate section modulus of the compression flange depending on the class of cross section. α_{LT} is equal to 0.34 for buckling curve 'b' [6].

2.3. BS5950 Part 5 [4]

The lateral-torsional buckling resistance moment *(M_b)* is given by

$$M_b = \frac{M_E M_Y}{\phi_B + \sqrt{\phi_B^2 - M_E M_Y}} \leq M_c \tag{4}$$

where, $\phi_B = \dfrac{M_Y + (1+\eta) M_E}{2}$ and $M_E = \dfrac{\pi^2 AED}{2 \left(L_E / r_y \right)^2} C_b \left\{ 1 + \dfrac{1}{20} \left(\dfrac{L_E}{r_y} \dfrac{t}{D} \right)^2 \right\}^{1/2}$

In the above, M_E is the elastic lateral buckling resistance moment of channel section beams. *A, D, t,* L_e and r_y are the area, overall depth, thickness, effective length and the radius of gyration about y axis. Eq.5 gives η, the Perry coefficient, and the moment distribution factor C_b may be assumed as unity or can be calculated. In other cases η is zero.

$$\text{when } L_e / r_y > 40 C_b \qquad \eta = 0.002 \left(\frac{L_e}{r_y} - 40 C_b \right) \tag{5}$$

3. FINITE ELEMENT ANALYSES

A simply supported cold-formed steel lipped channel beam under uniform bending moment was modeled in this research to investigate the lateral-torsional buckling behaviour at ambient temperature (Figure 1). Due to the symmetric loading and geometrical conditions, only half the span was modeled. S4R5 type four noded shell elements of 5 mm x 10 mm were used in the analyses. A series of tensile and compressive forces was applied to the nodes based on a triangular distribution to simulate a uniform bending moment distribution. The beam was modeled with idealized simply supported boundary conditions at the end supports, which allows major and minor axis rotations and warping displacement while preventing in-plane and out-of-plane translations and twisting.

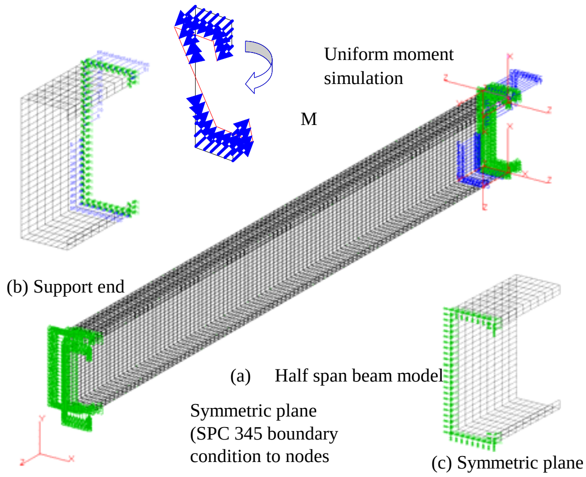

Figure 1: Finite Element Model of Lipped Channel Beam

The measured yield strength and elasticity modulus values at ambient temperature along with the developed equation for the stress-strain curve were used. Strain hardening model was used for high grade steels while using isotropic model for low grade steel. In the non-linear analyses, nominal imperfections and residual stresses were included. The buckling mode from the elastic buckling analysis was used to input the initial imperfection in the non-linear analysis. A maximum initial negative imperfection of *L/1000* was used since it yields the lowest failure moments compared to positive imperfections. Flexural residual stress distribution was used and assumed to be constant at $0.17f_y$ along the flanges and webs and $0.08f_y$ along the lips based on [7]. The finite element model was created by using MD/PATRAN and the analysis was conducted using ABAQUS. The developed model was validated using available numerical and experimental results as reported in [8].

Twelve cold-formed lipped channel beams made of 1.5 mm and 1.9 mm G450 steels and 1.55 mm and 1.95 mm G250 steels were selected for the detailed parametric study. Their cross section included compact plate elements based on AS/NZS 4600 [5]. This eliminated the occurrence of local buckling. Finite element analyses of these beams were conducted for spans varying from 700 mm to 5000 mm. The finite element analysis results of ultimate moment capacities for short and long span lipped channel beams are plotted in Figure 2 in a non-dimensional format of ultimate moment capacity (M_b/M_y) versus beam slenderness $(M_y/M_o)^{0.5}$. Further details of parametric study and the results are given in [8].

As seen in Fig. 2, the moment capacity of beams undergoing lateral-torsional buckling failures in the intermediate and elastic slenderness range is reduced below the elastic buckling capacity due to the presence of residual stresses and initial geometric imperfections. The beam capacity in the elastic buckling region (high slenderness) is approximated by the elastic lateral-torsional buckling capacity. The non-dimensional moment capacity curves were not dependent on the yield strength (G250 or G450).

4. COMPARISON OF ULTIMATE MOMENT CAPACITIES FROM FEA WITH PREDICTIONS FROM THE CURRENT DESIGN RULES

4.1. AS/NZS 4600 and Direct Strength Method (DSM)

Figure 2 compares the ultimate moment capacity results from FEA with the moment capacity curve based on the design rules in AS/NZS 4600 [5]. It shows that AS/NZS 4600 [5] design rules over-predict the moment capacity of intermediate and long span beams subject to lateral-torsional buckling and that the AS/NZS 4600 [5] moment capacity predictions are unsafe in both elastic and inelastic buckling regions. Further, according to AS/NZS 4600 the moment capacities of long spans beams with member slenderness, $\lambda > 1.336$, are equal to the elastic lateral-torsional buckling moments (M_u = M_o). However, Figure 2 shows that they are below the elastic buckling moments. Similar behaviour was observed by others [9] for other types of sections, and also for lipped channel beams [10] even at high slenderness values.

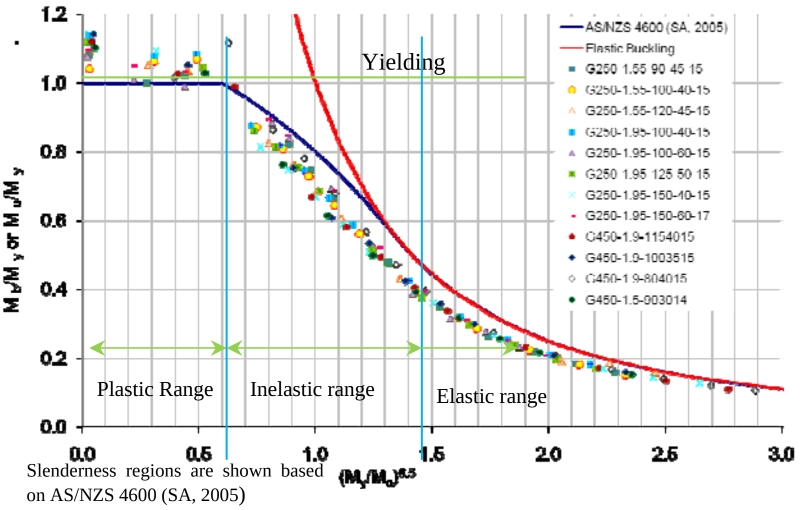

Figure 2: Ultimate Moment Capacity Curves from FEA and Comparison with AS/NZS 4600 Design Curves

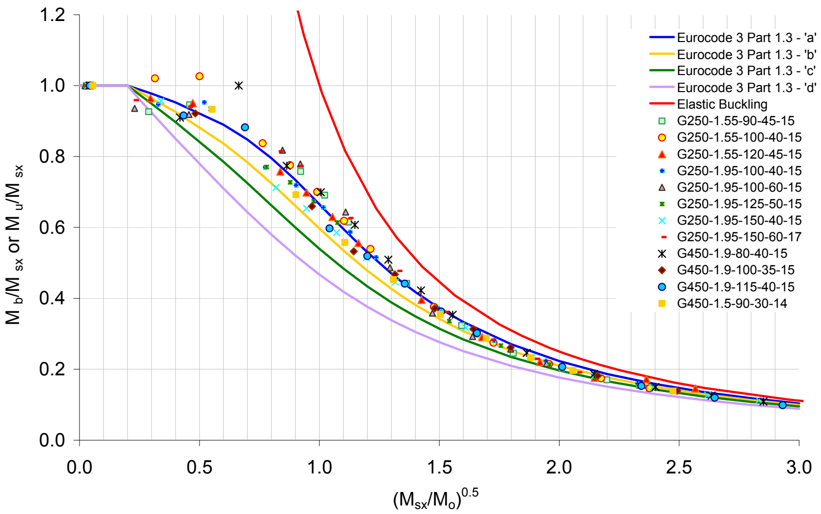

Figure 3: Comparison of FEA Results with Different Buckling Curves of Eurocode 3 Part 1.1 (ECS, 2005)

4.2. Eurocode 3 Part 1.3 [3]

As seen in Figure 3, Eurocode 3 Part 1.3 with its recommended buckling curve 'b' accurately predicts the moment capacities of beams with high slenderness values as defined by $(M_{sx}/M_o)^{0.5} \geq 1.5$, but is slightly overconservative for beams having intermediate slenderness values in the range of $(M_{sx}/M_o)^{0.5} < 1.5$ (Figure 3). However, the buckling curve 'a' in Eurcocde 3 Part 1.1 was found to be predicting accurate moment capacities in comparison to FEA results throughout the elastic and inelastic slenderness ranges.

4.3. BS5950 Part 5 [4]

The moment capacity predictions of BS5950 Part 5 were found to be higher than the FEA results and therefore it is concluded that BS5950 Part 5 [4] design equations for lateral-torsional buckling give unsafe predictions for cold-formed steel lipped channel beams [8].

5. DEVELOPMENT OF NEW DESIGN RULES FOR AS/NZS 4600 [5]

Based on the comparisons in the last section, it was identified that some modifications are needed for the AS/NZS 4600 [5] moment capacity equations. In this section a suitable modification is proposed. Since the design rules in NAS [11] and AS/NZS 4600 are identical, any modifications to AS/NZS 4600 design rules are equally applicable to the former. Three design options for new design method were proposed based on this study [8] and only the most suitable and accurate design method is given next. In the proposed design method, the critical moment (M_c) in Equation 1 can be calculated as follows,

For $\lambda_b \leq 0.6$
$$M_c = M_y \tag{6a}$$

For $\lambda_b > 0.6$
$$M_c = \underset{a}{0.9}\left[\sqrt{\lambda_b^4 + \underset{b}{2.05}} - \lambda_b^2\right] M_y \tag{6b}$$

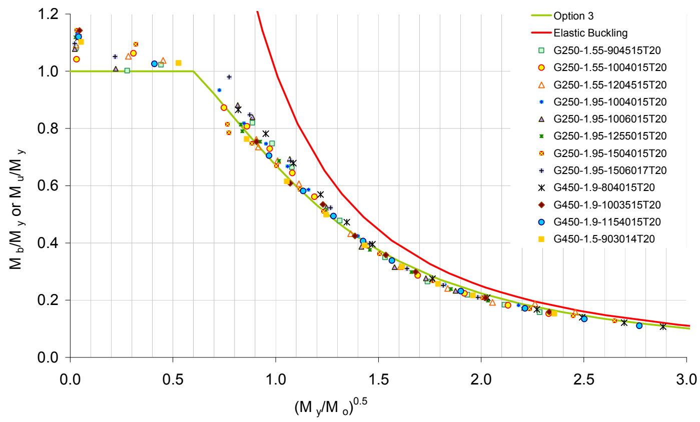

Figure 4: Comparison of FEA Results with New Design Beam Curve

Steel	$M_u/M_{pred.}$		
	Mean	COV	Φ
G250-1.55	1.004	0.067	0.902
G250-1.95	0.981	0.053	0.891
G450-1.5	0.944	0.018	0.868
G450-1.9	0.978	0.046	0.892
G250	0.990	0.060	0.896
G450	0.978	0.044	0.893
Overall	0.984	0.056	0.892

Table 1: Mean and COV Values of the Ratio of FEA to Predicted Capacities

Figure 4 compares the FEA results with the proposed moment capacity curve. It appears that the new equation accurately predicts the moment capacities of cold-formed steel beams in all the slenderness regions, and is therefore recommended for the design of cold-formed steel lipped channel beams subject to lateral-torsional buckling. Further, the mean, COV and capacity reduction factors of FEA to predicted moment capacities based on new design method given in Table 1 indicate the accuracy of the proposed design method.

The new equation is similar to the moment capacity equation in the Australian hot-rolled steel structures code, AS 4100 [12] and have many advantages in comparison to the AS/NZS 4600 design equations. Equation 6b provides a simple way of calculating lateral-torsional buckling capacities in the inelastic and elastic buckling regions. Another advantage of this proposal is that this equation can be modified by changing ‘a’ and ‘b’ as shown in Equation 6b to develop multiple beam curves. Moment capacities are dependent on the type of section and it is useful to develop multiple beam curves for different section types such as lipped channel sections, hollow flange sections, RHS, back to back C-sections etc. Trahair [13] has also shown the need to have multiple design curves to allow for varying section types, residual stresses and initial imperfections.

6. CONCLUSIONS

This paper has described a detailed parametric study on the member moment capacities of simply-supported cold-formed steel lipped channel beams subject to uniform bending about the major axis at ambient temperature using a validated ideal finite element model. The results showed that the current design rules in AS/NZS 4600 [5] are unsafe for lipped channel beams subject to lateral-torsional buckling and therefore new design equations are proposed. The new design equations were shown to predict the moment capacities accurately. The predictions of BS 5950 Part 5 [4] design equations were also found to be unsafe. However, the EC 3 Part 1.3 [3] design rules (EC 3 Part 1.1 with buckling curve ‘b’) were found to be accurate for cold-formed steel beams with high slenderness values while their predictions for beams with intermediate slenderness are over-conservative. Therefore the bucking curve ‘a’ is proposed with EC 3 Part 1.1 equations for cold-formed steel beams.

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