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The continuous strength method for lateral-torsional buckling of stainless steel I-beams

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ABSTRACT

Stainless steel is now widely used in construction as structural members in recognition to its unique beneficial properties such as corrosion resistance, higher strength, ductility, and negligible maintenance cost. Recent research on stainless steel has led to the evolution of a deformation based design rule, the Continuous Strength Method (CSM), which has been shown to perform well in predicting cross-sectional resistances but still requires considerable research to be used in predicting member resistances. The current paper proposes a new design method for lateral-torsional buckling (LTB) behaviour of welded stainless steel I-sections combining CSM design philosophy and traditional Perry-type concept used for column buckling. As part of the numerical study presented herein, nonlinear finite element (FE) models were developed and validated using available test results. Once the FE modelling technique was validated, a large number of reliable numerical results were generated to investigate effects of various factors on the resistance of members subjected to LTB. Obtained results showed that the cross-section slenderness $\bar{\lambda}_p$ and the non-dimensional proof stress e have significant influences on LTB resistance. Effects of e was appropriately incorporated by introducing a correction factor to modify $\bar{\lambda}_p$. As LTB curves were mostly affected by $\bar{\lambda}_p$, it was included in the equation for calculating imperfection parameter $\eta_{csm,LT}$, which is a key parameter to include member imperfections in Perry-type design equations. This new approach ensures appropriate utilization of strain hardening for stocky cross-sections and allows to avoid the complex process of calculating effective geometric properties for slender sections. All available test and generated numerical results were used to assess the performance the current European, the Australian and the proposed CSM based design rules for LTB. Comparisons clearly showed that the proposed approach performed significantly well in predicting the LTB response of stainless steel I-sections.

1. Introduction

Use of stainless steel in construction is gradually increasing as it offers high corrosion resistance, better ductility, improved fire resistance, higher strength and attractive appearance [1–3]. Material characteristics of stainless steel are distinctly different from ordinary carbon steel with a nonlinear stress-strain response and absence of any definite yield point requiring alternative treatment in structural design. Extensive research has been reported in recent times to develop better understanding of the structural response of stainless steel sections. Considerable experimental and numerical studies were conducted on stainless steel stub columns, beams and long columns [4–11]. These investigations outlined the shortcomings of available international design codes for stainless steel such as limiting the design stress to 0.2% proof stress and ignoring strain hardening properties. Hence, a strain based design method, the Continuous Strength Method (CSM) for stainless steel was proposed with appropriate recognition of its material response [12–14]. CSM has been modified during the last decade with the latest simple formulas proposed in [15–17]. CSM has been shown to produce accurate predictions for section resistances when compared against those obtained by the traditional design codes.

Initially, CSM was devised for section behaviour against individual actions such as compression and bending. In recent times, CSM guidelines have been proposed for more general loading conditions such as combined actions of compression and bending [18–24] and flexural buckling of columns [14,25–27]. However, lateral–torsional buckling (LTB), a collapse mode typical in open section members subjected to bending, is yet to be thoroughly addressed utilizing the benefits offered by CSM.

Welded sections are widely used in construction as they can be fabricated to suit specific design requirements, and hence, are one of the most commonly used shapes in structures. Unlike hollow sections,

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the geometric shape of I-section beams makes it vulnerable to instabilities due to LTB. Experimental investigations on stainless steel welded I-section beams presented in [28,29] were used in the current study to develop FE models to understand their structural response in LTB. Once validated against available test evidences, the developed FE models were used to conduct a comprehensive parametric study covering wide range of material and geometric properties. Obtained FE results were consequently used to develop CSM based buckling resistance equations to tackle LTB. This new set of equation is based on Perry type approach to maintain resemblance with EN 1993-1-4:2006 +A1:2015 [30]. Finally, performances of available design guidelines and that of the proposed CSM based design equations in predicting LTB resistance were thoroughly examined.

2. Existing design rules

2.1. EN 1993–1-4(2006)+A1(2015)

Eurocode EN 1993–1-4:2006 +A1:2015 [30] is the most updated design code among available guidelines for structural stainless steel. EN 1993–1-4 provided supplementary information specific for structural design of stainless steel, primarily based on those for ordinary carbon steel as specified in [31]; suggested formulas for lateral–torsional buckling are shown in Eqs. (1)–(4). The strength of a member subjected to bending was defined as $M_{b,Rd}$, which can be obtained by multiplying the cross-section bending resistance $M_{c,Rd}$ with an appropriate buckling reduction factor χ_{LT}. The first term, cross-sectional moment capacity $M_{c,Rd}$, is the product of major axis section modulus W_y, and material yield stress f_y. It should be noted that section modulus depends on cross-section types i.e. plastic section modulus $W_{pl,y}$ used for class 1–2 sections, elastic section modulus $W_{el,y}$ adopted for class 3, and effective section modulus $W_{eff,y}$ for class 4 cross-sections. Buckling reduction factor χ_{LT} is a function of the imperfection factor α_{LT} and the member slenderness λ_{LT}. In Eq. (3), the term α_{LT} $(\lambda_{LT} - 0.4)$ is collectively known as the imperfection parameter η, which is a common term in all EN 1993 codes for designing members against buckling. The safety factor γ_{M1} was taken as unity to evaluate the performance of the proposed design equations.

The buckling equation adopted in EN 1993 codes was based on *Ayrton-Perry* formulations, which were initially proposed for buckling design of column sections considering the effect of initial bow imperfection of a structural member. This was later calibrated by varying the imperfection factor α for calculating member resistances to suit various buckling induced cases and cross-section types. In the case of welded open sections of stainless steel subjected to LTB, the imperfection factor α_{LT} is specified as 0.76 according to EN 1993-1-4:2006 +A1:2015 [30]. While calculating the member slenderness λ_{LT} using Eq. (4), M_{cr} is the elastic critical moment for lateral-torsional buckling. Although EN 1993–1-4:2006 +A1:2015 [30] or EN 1993–1-1 [31] did not include any specific formula for calculating M_{cr}, it can be calculated according to the NCCI: SN003a [32] document.

$$M_{b,Rd} = \chi_{LT}\frac{M_{c,Rd}}{\gamma_{M1}} \tag{1}$$

Note:

$$M_{c,Rd} = W_{pl,y} \times f_y \quad \text{for class 1 and 2 cross-sections}$$

$$M_{c,Rd} = W_{el,y} \times f_y \quad \text{for class 3 cross-sections}$$

$$M_{c,Rd} = W_{eff,y} \times f_y \quad \text{for class 4 cross-sections}$$

$$\chi_{LT} = \frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^2 - \lambda_{LT}^2}} \leq 1 \tag{2}$$

$$\varphi_{LT} = 0.5 \times [1 + \alpha_{LT}(\lambda_{LT} - 0.4) + \lambda_{LT}{}^2] \tag{3}$$

$$\lambda_{LT} = \sqrt{M_{c,Rd}/M_{cr}} \tag{4}$$

EN 1993 codes allow use of plastic section properties for calculating cross-section resistance, which may be deemed sufficient for stocky type carbon steel cross-sections as they reach a plastic deformations plateau just followed by yielding of the material. Contrast to that, stainless steel stocky cross-sections exhibit extensive nonlinear strain hardening, which has not been appropriately considered in EN 1993 resulting in conservative predictions for cross-sectional resistance. For slender type cross-sections, interactions among the cross-section elements are not rationally incorporated, and slenderness of a cross-section is defined based on the most slender element of the cross-section. Thus conservative predictions were also observed in case of resistance predictions for slender cross-sections. To compensate for the observed conservatism in cross-sectional resistances, the buckling curves were calibrated to obtain member resistances accurately. In this study, performance of the lateral-torsional buckling curves presented in EN 1993–1-4:2006 +A1:2015 [30] for stainless steel members will be thoroughly investigated.

2.2. Australia and New Zealand Standard –AS/NZS 4673 (2001)

Stainless steel design rules presented in Australian and New Zealand AS/NZS 4673 [33] and those proposed by the American Society of Civil Engineering in SEI/ASCE8-02 [34] are very similar. Unlike EN 1993 codes, AS/NZS code uses a tangent modulus approach to determine the critical moment $M_{cr,0}$ of the member segment subjected to lateral–torsional buckling. Eq. (5) presents a rearranged form for calculating $M_{cr,0}$ derived from Eq. 3.3.3(4) of the AS/NZS 4673 [33]. Bending coefficient C_b is taken as unity in the current study as uniform bending moment is applied throughout the member length. Other parameters in Eq. (5) are as follows –E_t is the tangent modulus, E_0 is the Young's modulus, G_0 is the shear modulus, and L_y and L_t are respectively the effective lengths for lateral bending and twisting, and lastly, I_z, J, C_w are the second moment of area about the minor axis, torsional constant and warping constant of the cross-section, respectively,.

Here, tangent modulus, E_t requires iteration of the inelastic buckling stress f_{ib}, and thus the whole process of calculating critical moment $M_{cr,0}$ becomes a repetitive procedure using Eqs. (5)–(7). In Eq. (6) for determining E_t, the term n is the Ramberg-Osgood parameter for strain hardening.

$$M_{cr,o} = C_b\frac{E_t}{E_0}\sqrt{\frac{\pi^2 E_0 I_z}{L_y{}^2}\left(G_0 J + \frac{\pi^2 E_0 C_w}{L_t{}^2}\right)} \tag{5}$$

$$E_t = \frac{f_y E_0}{f_y + 0.002nE_0(f_{ib}/f_y)^{n-1}} \tag{6}$$

$$f_{ib} = \frac{M_{cr,o}}{W_{el,y}} \tag{7}$$

Once the inelastic buckling stress f_{ib} is determined, effective section modulus $W_{e,y}$ can be calculated using the effective cross-sectional properties. The nominal moment capacity $M_{b,Rd}$ of a member segment can be determined using Eq. (8), where $W_{e,y}$ is the effective section modulus, $M_{cr,o}$ is the critical moment and $W_{el,y}$ is the elastic section modulus for gross-section. The resistance factor φ_b will be taken as unity when evaluating the performance of the design equation.

$$M_{b,Rd} = \phi_b M_{cr,o}\frac{W_{e,y}}{W_{el,y}} \tag{8}$$

Australian standard AS/NZS 4673 [33] restricts the cross-sectional strengths at elastic section properties, and does not consider material plasticity or strain hardening, which in some cases produce more conservative predictions compared to EN 1993 codes. However, tangent-modulus approach adopted in AS/NZS 4673 incorporates the Ramberg-Osgood parameter n for strain hardening, which recognises the