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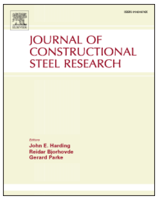

Local-flexural interactive buckling of standard and optimised cold-formed steel columns

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ABSTRACT

This paper aims to study the interaction of local and overall flexural buckling in cold-formed steel (CFS) channels under axial compression. Detailed nonlinear FE models were developed and validated against a total of 36 axial compression tests on CFS plain and lipped channel columns with pin-ended boundary conditions. The numerical models incorporated the non-linear stress-strain behaviour of CFS material and enhanced properties of cold-worked corner regions obtained from coupon tests. The effects of initial geometric imperfections of the specimens measured by a specially designed set-up with laser displacement transducers were also taken into account. The developed FE models produced excellent predictions of the ultimate strength of the specimens obtained from experimental tests. The validated FE models and experimental results were then used to assess the adequacy of the effective width method in Eurocode 3 (EC3) and Direct Strength Method (DSM) in estimating the design capacity of a wide range of conventional and optimised design CFS channel column sections. The results indicate that Eurocode 3 provides conservative predictions (on average 21% deviation) for the compressive capacity of plain and lipped channel sections, while in general DSM predictions are more accurate for lipped channels. A comparison between FE predictions and tested results show that geometric imperfections can change the FE predictions by up to 20% and 40%, respectively, for lipped and plain channel columns, while the strain hardening effect at the rounded corner regions of the cross-sections is negligible. The results also confirmed that the proposed numerical model is able to provide a consistent and reliable prediction on the efficiency of a previously proposed optimisation methodology.

Notation

b, b_e Gross and effective width of the plate
c Lip width
h Web height
t_{eff} Effective thickness
L_e Length of the column
I Coil width of the steel sheet
A_s Area of the edge stiffener
I_s Effective second moment of area of the stiffener
f_y Material yield stress
E Young's modulus
ψ Stress ratio
ρ Reduction factor on the plate width
σ_{cr} Elastic local buckling stress
$\sigma_{cr,\ s}$ Elastic critical buckling stress for an edge stiffener
K Spring stiffness per unit length
χ_d Reduction factor for flexural buckling of the stiffener
λ_p, λ_d, λ Local, distortional and global buckling slenderness ratio in effective width method
$\lambda_{p,\ red}$ Updated λ_p in each iteration
λ_l, λ_{dl}, λ_c Non-dimensional local, distortional and global buckling slenderness in the DSM
N_{Ed} Design value of the compression load
$N_{b,\ Rd}$ Design buckling resistance of a compression member
$M_{b,\ Rd}$ Design pure bending moment resistance around weak axis
e_N Shift of centroid
ΔM_{Ed} Additional bending moment due to shift of centroid
P_y Compressive yield load
P_{crl}, P_{crd}, P_{cre} Elastic critical force for local, distortional and global buckling modes
P_{nl}, P_{nd}, P_{ne} Axial strength for local, distortional and global buckling modes, respectively
P_n Ultimate axial strength of the column
P_{u1}, P_{u2}, P_{u3} Predicted axial strengths considering the effects of strain hardening of the material in the corner regions, measured geometric imperfections, and both.

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1. Introduction

In common practice, cold-formed steel (CFS) structural elements have traditionally been employed as secondary load-carrying members such as stud walls, roof purlins, wall girts and cladding. However, in a more recent trend, CFS members are also increasingly being employed as primary structural elements in low- to mid-rise multi-storey buildings [1] and CFS portal frames with short to intermediate spans [2,3]. Compared to hot-rolled members, CFS thin-walled members offer several advantages, such as a high strength for a lightweight, a relatively straightforward manufacturing process, a high flexibility in obtaining various cross-sectional shapes, and an ease of transportation and faster construction. However, as a result of the limitations of the manufacturing process, CFS components usually have <6–8 mm thickness, which makes them susceptible to local, distortional and global buckling, as well as their interactions. The typical buckling modes of a lipped channel are illustrated in Fig. 1.

The theoretical aspects of local-flexural interactive buckling were first established by Van der Neut [4] on the basis of an elastic idealized column with two flanges supported along both longitudinal edges by infinitely thin webs. This early work, in combination with Van der Neut's later paper [5], demonstrated that the capacity of CFS columns is sensitive to both local and global imperfections, especially when the critical stresses of both buckling modes are of the same level. However, due to the inherent weaknesses of thin-walled cross-sections and their complex buckling modes, the accurate prediction of the buckling and post-buckling behaviour of CFS elements is relatively challenging. Finite Element Analysis (FEA) has been widely used in the past to predict the non-linear behaviour of CFS elements. Compared with physical experiments, FEA is relatively inexpensive and time efficient, especially when a parametric study of cross-section geometry is involved. In addition, FEA is more suitable and convenient for studies involving geometric imperfections and material nonlinearity of structural members, which could be difficult to investigate through physical tests.

In one of the early attempts, Young and Yan [6] developed a nonlinear FE model to investigate the compressive strength of fixed ended CFS columns, using four node shell elements with five degrees of freedom per node. Reduced integration was used (SR4) in combination with linear perturbation analysis ‘BUCKLE’ to incorporate imperfection effects. Based on experimental results on CFS fixed-ended lipped channel columns, Young [7] used a nonlinear inelastic FE model to investigate the effect of inclined edge stiffeners on ultimate axial capacity. Similarly, Yan and Young [8,9] experimentally and numerically studied the ultimate capacity of fixed-ended CFS channel columns with complex stiffeners. SR4 element type in ABAQUS [10] was used by taking into account initial geometric imperfections and material non-linearity. In another study, Zhang et al. [11] conducted an experimental test program on pin-ended CFS columns with perpendicular and inclined edge stiffeners and developed FE models using four-node shell element type with six degrees of freedom at each node in ANSYS [12]. The rigid region at each end of the column elements was modelled with a reference point, where rotations around both strong and weak axis of the end sections were allowed. Wang et al. [13] conducted a series of experimental tests on pin-ended columns with complex cross-sectional edge and intermediate stiffeners and the results were compared with the FE models similar to one proposed by Zhang et al. [11]. In a recent study, Ayhan and Schafer [14] used an experimentally verified numerical model in ABAQUS [10] to obtain moment-rotation curves and characterize the backbone response curve of CFS members in monotonic bending. Based on both experimental and numerical results, a series of new local/distortional slenderness based design equations were proposed to provide a rapid estimation of the buckling and post-buckling behaviour of CFS members.

To obtain more efficient design solutions, Ma et al. [15] and Ye et al. [16] developed a practical optimisation framework for CFS channel cross-sections in compression or bending based on the effective width method adopted in Eurocode 3 [17–19]. In their framework, the plate slenderness limits and the limits on the relative dimensions of the cross-sectional components set by the Eurocode as well as a number of construction and manufacturing constraints were taken into account. The results showed that, in general, optimised CFS sections possess relatively higher axial and flexural strength compared to other standard prototypes. However, even though available design equations developed in Eurocode 3 are well accepted for the calculation of strength of CFS members, their ability to estimate the increasing/decreasing trend in optimisation process is still questionable. On the other hand, the adequacy and reliability of the optimisation method adopted by Ma et al. [15] and Ye et al. [16] should be validated by experimental results or accurate numerical models before they can be widely used in practice.

This paper aims to investigate the local-flexural interactive buckling behavior and ultimate capacity of CFS standard and optimised plain and lipped channel columns by developing detailed FE models in ABAQUS [10]. The results of a companion experimental investigation including 36 tests on CFS channel columns [20,21], which were all failed by the interaction of local instability and flexural buckling about the minor axis, are used to validate the FE models. Compared to previous studies, the main advantage of the developed models is to incorporate the non-linear stress-strain behaviour of CFS material and enhanced properties of cold-worked corner regions (based on coupon tests) as well as the measured initial geometric imperfections. The validated models are then used to assess the adequacy of Eurocode 3 design guidelines [17–19] and Direct Strength Method (DSM) for the design and optimisation of CFS columns considering local/distortional and global buckling modes.

2. Eurocode 3 design procedure

Prior to the description of the numerical study, a brief induction is presented here to explain the EC3 design guidelines to consider local, distortional and global buckling, their interaction and inelastic reserve around minor axis in the compressive strength of CFS members.

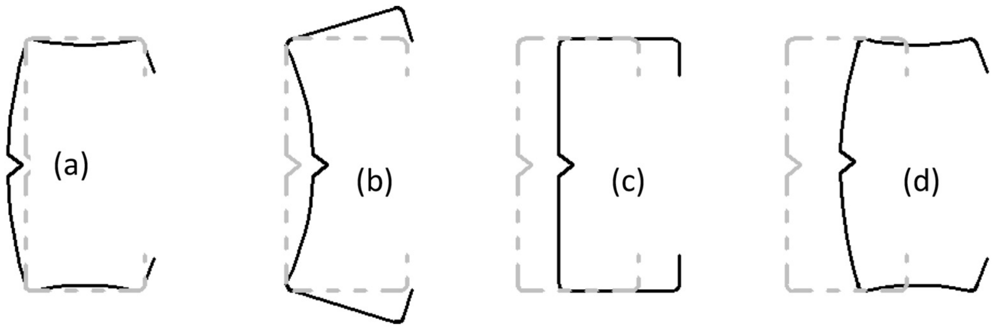

Fig. 1. Buckling modes of a lipped channel: (a) local, (b) distortional, (c) global and (d) local-global interactive modes.