Thin-Walled Structures 79 (2014) 62–70

Contents lists available at ScienceDirect

Thin-Walled Structures

journal homepage: www.elsevier.com/locate/tws

Strength design curves and an effective width formula for cold-formed steel columns with distortional buckling

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ARTICLE INFO

Article history:
Received 16 July 2013
Received in revised form
4 December 2013
Accepted 3 February 2014
Available online 22 February 2014

Keywords:
Cold-formed steel columns
Distortional buckling
Post-buckling strength capacity
Strength design curves
Effective width formulas

ABSTRACT

Distortional buckling mode of cold-formed steel thin-walled member is an unstable behavior, and in some cases it may govern the load-carrying capacity of the member. The source, evolution and performance of the formulas and test data for the two strength design curves developed by Hancock are studied, for predicting the load-carrying capacity in the distortional mode. A proposed strength design curve based on available test data and Hancock's strength design curves are then compared with the current design methods, the Direct Strength Method and the Effective Width Method, which are incorporated in the "North American specification for the design of cold-formed steel structural members" (AISI-NAS: 2007), "cold-formed steel structures" (AS/NAS 4600: 2005), and the Chinese "Technical specification for low-rise cold-formed thin-walled steel buildings" (JGJ 227-2011). The results indicate that the current design standards adopted the two strength design curves for the DSM and EWM, but they have some differences at the partial extent. A novel formula is proposed for dealing with this problem. The range of applicability of the proposed strength equation is extended from that in AS/NZS 4600 and is shown to be more accurate than AS/NZS 4600 when compared with that in the NAS S100.

1. Introduction

Cold-formed steel applications continue to advance in three primary categories: framing, metal buildings, and racks. The behavior of cold-formed steel structures can be complicated due to the thin-walled nature of the sections [1]. Cold-formed steel thin-walled members with open cross-sections are highly susceptible to structural instabilities, such as local (L), Euler (E) and distortional (D) buckling; in some cases, the buckling mode can play a dominate role for the structural behavior and the ultimate strength. It is well known that thin-walled members display stable local and global post-buckling behaviors with high and low post-critical strength capacity, respectively [2]. Distortional post-buckling behaviors fit between the previous two behaviors and exhibit a non-negligible asymmetry with respect to the flange-stiffener motion (outward or inward). Depending on the member geometry (length, cross-section shape and dimensions) and end support conditions, any of three buckling modes may be critical; in some cases, distortional buckling mode may govern the load-carrying capacity of the member.

The behavior of distortional buckling of the cold-formed steel thin-walled sections was reported many years ago. However, after a long period of time, local buckling and flexural–torsion buckling and local–global interaction attracted considerable attention [3]. Distortional buckling was treated in light of the methods of previous buckling modes; therefore, distortional behavior was named "stiffener buckling mode" [4–7] and "local–torsion buckling mode" [8]. In 1985, distortional buckling was first discussed by Hancock [9] for steel storage rack sections. Hancock presented simple design charts for computing the buckling stress. Simplified formulas of computing the elastic distortional buckling stress for sections with edge-stiffened elements were provided by Lau and Hancock [10]. After performing a series of tests in the inelastic distortional buckling range, Lau and Hancock [11] presented in a preliminary set of design curves based on the Johnston parabola for distortional buckling. The strength design curve did not allow for post-buckling behavior in the distortional mode. In 1992, Kwon and Hancock [12] tested channel sections with and without intermediate web stiffeners composed of high-strength steel with a yield stress of 550 MPa. The results showed that distortional mode had a significant post-buckling strength reserve, yet had a

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http://dx.doi.org/10.1016/j.tws.2014.02.004
0263-8231

Nomenclature

A, A_g	area of the full cross-section
A_{cd}	effective distortional buckling area
b_e	effective flat width of the compression element
b	flat width of the compression element
f_{cr}, f_{crl}	elastic local buckling stress
f_{crd}, f_{de}	elastic distortional buckling stress
f_{di}	inelastic distortional buckling stress
f_m	maximum strength test values
f_u	maximum design stress
f_y	yield stress
M_{de}, M_{crd}	elastic critical moment for distortional buckling
M_m	test maximum moment capacity
M_{nd}	nominal flexural strength for distortional buckling
M_u	ultimate moment capacity
M_y	moment causing the initial yield at the extreme compression fiber of the full section
P_{crd}	critical elastic distortional column buckling load
P_{nd}, P_{cd}	nominal column strength for distortional buckling
P_y	yield load for the column
S_g, Z_f	gross section modulus referenced to the fiber at first yield
λ	global slenderness value
λ_d, λ_{cd}	distortional slenderness value

lower capacity than the local mode. Based on prior test results, Kwon and Hancock [12] developed two strength design curves for the distortional buckling mode for predicting the load-carrying capacity of these members. Kwon and Hancock [13] developed a nonlinear elastic spline finite strip method to predict the post-buckling behavior of thin-walled sections for the tested channel columns. Hancock et al. [14] provided illuminating experimental evidence of distortional instability of cold-formed steel members with various cross-section shapes. They used numerical programs (i.e., semi-analytical and spline finite strip methods) to evaluate the distortional buckling and post-buckling behavior. They again suggested the two strength design curves to estimate the ultimate load-carrying capacity and design the members of cold-formed steel undergoing a distortional failure. Hancock [15,16] modified the Lau and Hancock elastic buckling stress formulas for sections in compression to apply them to distortional buckling in flexure and presented design curves for determining the post-distortional strength. A series of studies on distortional buckling by Hancock and his research team attracted considerable attention. Schafer and Peköz [17,18] firstly mentioned the Direct Strength Method (DSM) to explore cold-formed steel design for beams based on a large database of sections undergoing local and distortional buckling. For columns with local buckling, the strength curve was selected to be similar to the curve previously discovered for beams. For distortional buckling of columns, one of the two strength curves proposed by Hancock et al. was adopted by Schafer [19,20]. As Schafer stated, the beginning of the DSM for columns can be traced to research of distortional buckling at the University of Sydney. In particular, Hancock et al. collected the research and demonstrated that for a large of variety of cross-sections the measured compressive strength in a distortional failure correlated well with the slenderness in the elastic distortional mode [21,22]. Elastic critical buckling stresses are required by the DSM approach, and those solutions have to be obtained by advanced computational analyses. Elastic buckling analysis software THINWALL (Papangelis and Hancock [23]) and CUFSM (Schafer [24]), elastic buckling and vibration analysis software GBTUL (Silvestre et al. [25]), which were based on finite strip theory and generalized beam theory respectively were developed to analyze the buckling half-wavelength and corresponding buckling stresses.

One of the two strength design curves improved by Hancock et al. was used by the DSM for columns with a distortional mode, which had been incorporated in the current versions of the Australian/New Zealander (AS/NZS 4600: 2005) [26] and North American (AISI-NAS 2007) [27] cold-formed steel design specifications. The second of the two strength design curves was adopted by the Effective Width Method (EWM), which also had been incorporated in the Australian/New Zealander (AS/NZS 4600: 2005) [26] and Chinese code "Technical specification for low-rise cold-formed thin-walled steel buildings" (JGJ 227-2011) [28]. According to recent researches by Kwon et al. [29], Hancock et al. [30] and Young et al. [31], the interaction of distortional buckling with other modes is detrimental to the load-carrying capacity of these members. To predict the reduced load, the DSM equation of the distortional mode was used in the novel design approach. Hancock's strength design curves, based on the same test data, were derived from the Johnston [32] parabola and the Winter formula respectively. However, the two strength design curves in DSM and EWM, have some differences in partial range applicability. The range of applicability for the strength curve employed in the EWM is shorter than the curve employed in DSM.

The main purpose of this study is to compare the performance of the two strength design curves employed in current design standards for cold-formed steel members with a distortional mode in compression. A novel strength design curve with an extended range of applicability for the current EWM is provided. The novel design curve is based on test results performed on a series of sections at the University of Sydney and conducted on a set of high strength and mild steel for channel columns by Young et al. [31] in the recent years. Lastly, a novel design formula of the EWM, using the proposed strength design curve and the current EWM shape, is presented to predict the ultimate load-carrying capacity of cold-formed steel thin-walled columns with distortional buckling.

2. Strength curves and design method on distortional buckling

2.1. Strength design curve 1 of Hancock

The Johnston [32] parabola of cold-formed steel columns in compression is expressed as

$$\text{for} \quad \lambda \le \sqrt{2}\, f_u = f_y\left(1-\frac{\lambda^2}{4}\right) \tag{1a}$$

$$\text{for} \quad \lambda \ge \sqrt{2}\, f_u = \frac{f_y}{\lambda^2} \tag{1b}$$

Lau and Hancock [11] conducted a set of tests for hat, channel and storage rack sections with yield stresses in the range of 200–480 MPa. As the distortional buckling stress was greater than half the yield stress, significant post-buckling strength capacity was not observed in the results. Therefore, Lau and Hancock used a design formula similar to the formula used by Chajes and Winter for inelastic flexural–torsional buckling to predict the inelastic distortional buckling stress. The proposed formula, based on the Johnston parabola, is