Journal of Constructional Steel Research 110 (2015) 163–169

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

Design expression for web shear buckling of box sections by accounting for flange restraints

Osama Bedair *

623-5330 58A Street, Red Deer, Alberta, T4N 3W1, Canada

ARTICLE INFO

Article history:
Received 11 August 2014
Accepted 26 February 2015
Available online 10 April 2015

Keywords:
Cold formed steel
Stiffened plates
Channel sections
Steel design
Shear panels
Buckling

ABSTRACT

This paper presents a new analytical expression for computing the web shear buckling stress with partial flange rotational restraints. The derived expression is suitable to be used in practice for hand calculation and avoids excessive efforts required to perform numerical analysis using finite element or finite strip methods. The material savings resulting from using the proposed design expression are also illustrated. Current design provisions ignore the flange rotational restraints effect in determining the web shear buckling stress for box sections. A comparison is made with AISI, CSA-S16, Eurocode 3 and AASHTO provisions for the limiting conditions. It is also shown that the shear buckling stress may vary by 40% with the current design expressions. Numerical validation of the proposed expression with semi-analytical finite strip method is also presented. The influence of the flange geometric properties on the web buckling stress is also highlighted.

1. Introduction

Box sections are widely used in commercial buildings, bridges, marine structures and industrial facilities. Box sections are fabricated in practice by assembling plate elements as shown in Fig. 1a or in some cases by connecting two cold-formed C-sections as illustrated in Fig. 1b. The second procedure is becoming popular in building construction, to increase the load carrying capacity for longer spans. Web shear buckling of the box section is a common failure mode in sections with large depth to width ratios. It is common practice to stiffen the compression flange in the longitudinal and transverse directions to enhance the compressive buckling capacity. Vertical stiffeners are also used to enhance the web shear capacity. It is common practice to space the web vertical stiffeners equally to simplify the fabrication process. Using vertical stiffeners allows the designer to reduce the web thickness, thus resulting in material savings. The rigidity requirements for the longitudinal stiffeners of box girder flanges are given by several design codes such as AASHTO specifications [1]. The validity of this requirement is, however, limited to bridge constructions and the accuracy has also been questioned in several investigations.

Considerable research efforts have been carried to identify various parameters affecting strength and serviceability limit states. Various analytical, numerical and experimental procedures were developed aimed to provide a better understanding of structural behaviors. An early work by Vlasov [2] forms the basis of modern analytical models used in practice to analyze box sections. The structure, in this case, is modeled as one dimensional beam element with either single or multiple spans to calculate the longitudinal stresses and displacements. More refined models then evolved utilizing advanced numerical analysis procedures such as finite element or finite strip methods to determine the two dimensional or three dimensional stresses and displacements. Examples of finite element formulations for the analysis of stiffened box sections were presented in references [3–10]. Finite strip formulations were also presented in references [11–14].

Bedair [15–31] presented semi-analytical procedures for static and dynamic analyses of stiffened webs and flanges. Various stiffener profiles (rectangular, Tee, Zee) were used in the investigation. The influence of the flange/web proportions on the structural performance was highlighted. Graphs were presented for several box sections to provide cost-effective design space that can be effectively used in the industry to optimize the section design. Hsu and Juang [32] investigated the influence of internal bracings on the local buckling behavior of box sections. They used a pair of braces bolted to both flanges and webs at plastic hinge locations. Experimental information was used to verify the effectiveness of the proposed internal bracing system. Cetinkaya et al. [33] investigated the behavior of box sections under variable bending moment and axial forces. Zheng et al. [34] studied the ductility of a compressive flange of short steel box columns with and without longitudinal stiffeners. Ramkumar et al. [35] investigated free vibration and damping of thin-walled box sections using the finite element method. Vo and Lee [36] investigated the flexural–torsional behavior of composite box beams subjected to vertical and torsional load. Zhu and Michael [37] investigated the effect of shear lag in thin-walled box girder bridges with large width-to-span ratios using the finite element method. Other studies in references

* Tel./fax: +1 403 356 1733.
E-mail address: obedair@gmail.com.

http://dx.doi.org/10.1016/j.jcsr.2015.02.018
0143-974X/

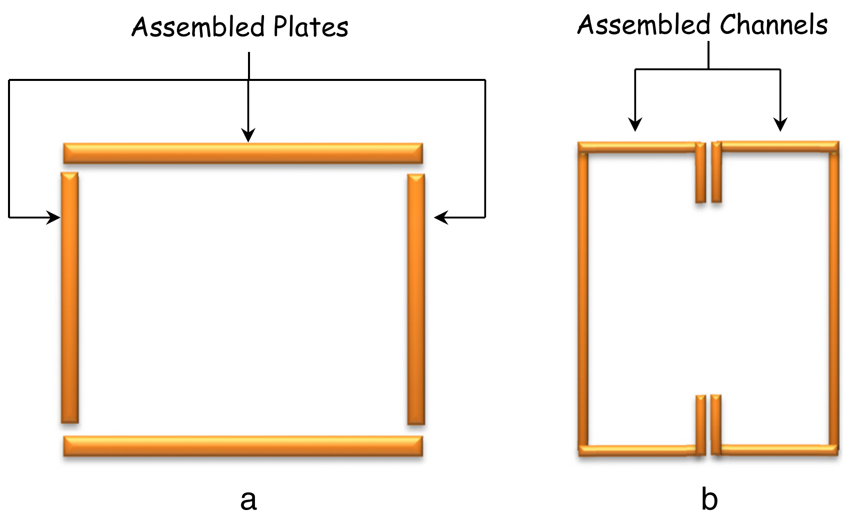

Fig. 1. Examples of assembled box sections.

[38–41] illustrated the influence of shear lag under concentrated or uniform loads.

Local buckling of box sections filled with concrete has also received the attention of several researchers. Although the concrete fill increases the cost of the member, it acts as restraining media to the plates and prevents inward buckling of the box members, thus increasing compressive strength. The buckling coefficient may increase by 36% for the simply supported boundaries. Shanmugam et al. [42] employed effective width procedure to predict the load carrying capacity of thin-walled steel tubes with concrete fill subject to biaxial loadings. Uy and Bradford [43] used the finite strip method to determine the buckling stress for various boundary conditions. A sinusoidal function is used for the longitudinal displacement and a cubic polynomial for the transverse displacement. Empirical studies were also performed for local buckling strength of steel tubes filled with concrete by Sakai et al. [44] and Wright [45].

Several experimental studies were conducted to evaluate the behavior of box girders. Spence and Morley [46] performed tests on box girders under different combinations of symmetrical and anti-symmetrical loads. Rasmussen and Baker [47] studied experimentally the ultimate load-carrying capacity and failure mechanisms of thin-walled box section beams subject to eccentric loads. Heins and Lee [48] reported field tests of a two-span curved steel single-box girder bridge. Reyes and Guzmán [49] performed an experimental investigation to study the behavior of box sections composed of two welded C-section members under uniform compression. Usami and Fukumoto [50] presented experimental results for local and overall buckling of welded box sections fabricated from high strength steel.

Limited literature addressed the shear buckling of box sections. Much of the investigations focused on developing numerical or empirical analysis procedures for box sections under compression or bending. The objective of the paper is to present an analytical closed form expression for predicting the shear buckling load of box sections by accounting for the flange rotational restraints. The influence of the flange width and thickness and rotational restraints on the web shear buckling is highlighted. The paper provides the designers with a design expression that can be used in practice without the need to perform finite element or finite strip analyses.

2. Current design expressions

The author could not find any analytical expression in the published literature to determine the shear buckling load of webs of box sections accounting for the flange rotational restraints. The common procedure used in practice is to idealize the web in isolation by assuming hinged boundary condition along the web/flange junction lines. In doing so, the rotational restraints imposed by the flanges are ignored. The widely known expression for computing the elastic shear buckling stress of a rectangular web panel is given by:

$$\tau = K_w \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t_w}{b_w}\right)^2 \quad (1)$$

where E is the modulus of elasticity, v is Poisson's ratio and t_w and b_w are the web panel thickness and depth, respectively. The shear buckling coefficient (K_w), depends upon the boundary conditions and the aspect ratio of the web panel. The actual boundary condition at the web/flange junctions is between simply supported and fixed limits. Current design codes conservatively assume simply supported edges. For example, the North American Cold Formed Steel Design Specification, AISI [51], CSA-S136-07 [52] and Canadian Standards Association for Limit States Design of Steel Structures CAN/CSA-S16 [53] provide shear buckling coefficients based on simply supported edges, ignoring the rotational restraints of the flanges. For unreinforced webs, a constant value of $K_w = 5.34$ is used. For webs with transverse stiffeners, the following equations are used:

$$K_w = 4 + \frac{5.34}{(a/b_w)^2} \quad for \quad (a/b_w) \le 1 \quad (2)$$

$$K_w = 5.34 + \frac{4}{(a/b_w)^2} \quad for \quad (a/b_w) > 1 \quad (3)$$

where (a) is the distance between the two adjacent transverse stiffeners.

The Structural Stability Research Council (SSRC) [54], provides the following expressions for web panels clamped along the longitudinal edges and simply supported on the opposite edges:

$$K_w = \frac{8.98}{\alpha^2} + 5.6 - 1.99\alpha \quad for \quad (\alpha) \le 1 \quad (4)$$

$$K_w = 8.98 + \frac{5.61}{\alpha^2} - \frac{1.99}{\alpha^3} \quad for \quad (\alpha) \ge 1 \quad (5)$$

where $(\alpha) = (a/b_w)$. The above expressions (4, 5) were developed from the numerical results of Cook and Rockey [55] using polynomial curve fitting technique.

Eurocode 3 [56] provides a general shear buckling equation for webs with longitudinal and transverse stiffeners, given by:

$$K_w = 4 + \frac{5.34}{(a/b_w)^2} + k_{st} \quad for \quad (a/b_w) \le 1 \quad (6)$$