Thin-Walled Structures 98 (2016) 578–591

Contents lists available at ScienceDirect

Thin-Walled Structures

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

Full length article

The shear strength of end web panels of plate girders with tension field action

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A R T I C L E I N F O

Article history:
Received 7 May 2015
Received in revised form
27 October 2015
Accepted 3 November 2015
Available online 9 November 2015

Keywords:
Shear tests
Plate girders
Web panel
Direct strength method (DSM)
Shear buckling
Post-buckling strength

A B S T R A C T

This paper describes a series of shear tests of plate girders fabricated from steel plates with a nominal yield stress of 235 MPa for the web and 315 MPa for flanges. The ultimate shear strength and performance of end web panels with a single double-sided transverse stiffener at support of plate girders undergoing a significant tension field action were investigated experimentally and theoretically. According to shear test results, post-buckling strength in the shear buckling mode has a significant effect on the ultimate shear strength of end web panels of plat girders. Design shear strength formulae for the direct strength method (DSM) for end web panels of plate girders were developed based on shear test results. The results verify that DSM shear curves can accurately predict the ultimate shear strength of web panels of plate girders showing a significant post-buckling behavior in the shear buckling mode.

1. Introduction

The web panels of plate girders show shear elastic or inelastic shear buckling and post-buckling behavior according mainly to its depth to thickness ratio and distance between transverse stiffeners to depth ratio. The post-buckling behavior of web panels subjected to shear loading was first reported in Wilson [1]. Since then, Wagner [2] employed the theory of diagonal tension field action, and many researchers examined the tension field action of plate girders until the 1970s [3–11]. Some studies [3–5] have assumed the diagonal tension field to develop in limited areas of the web, whereas others [6–8] have assumed it to develop across the whole web. However, the fundamental assumption is that the compression stress in the web does not increase after an elastic shear buckling. AISC specifications [12] adopt Basler's theory [3] to account for the post-buckling strength reserve to predict design shear strength, and AASHTO [13] adopts a similar formula. BS5400 [14] has a shear resistance formula based on the Cardiff model [5]. Both these models reasonably predict the post-buckling strength of the web of plate girders with a ratio of stiffener spacing to the web depth of less than 1.5.

In the 1990s, Lee and Yoo [15] found that these models produce fairly conservative predictions for web panels with a web-depth ratio of 3.0. Through a series of analytical and experimental studies [15–19], Lee and his colleagues showed that conventional models are not accurate and that vertical stiffeners and flanges do not behave as anchors, which is in contrast to the assumption of Basler's [3] model. They proposed a shear cell analogy and showed that all post-buckling forces are self-equilibrated within the web panel and thus that the end web panel also has a post-buckling strength reserve. Recently, based on numerous studies of the behavior of transverse stiffeners in tension field action [17,20–23], provisions for transverse stiffeners in the AISC specifications [24] have changed. The area requirement is no longer specified, but the demand on flexural rigidity has increased.

The direct strength method (DSM), first developed for cold-formed steel members by Schafer and Pekoz [25], has been verified to be designer-friendly and reliable in comparison to the conventional effective width method (EWM), which has been used in thin-walled steel section designs for more than seven decades. Recently, the DSM has been adopted by the North American Specification (NAS) (AISI standard, 2013) [26] and Australian/New Zealand Cold-Formed Steel Structures Standard AS/NZS 4600 [27]. However, the development of the DSM for welded steel members has commenced only recently. Kwon et al. [28–30] applied the DSM to welded section columns based on compression test results for H-, C-, RHS-, and CHS-section columns. A design strength formula for the DSM has been proposed for stiffened plates [31], and this method has been extended to predict the design flexural strength of welded H-section columns based on simple bending tests [32] and the design strength of the steel skin of concrete-filled tubular section columns [30]. A direct strength formula for

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http://dx.doi.org/10.1016/j.tws.2015.11.002
0263-8231/

Notations

A_w	area of web
a	distance between transverse stiffeners in the web
b	plate width
b_f	flange width
C_v	ratio of shear buckling stress to shear yield stress
d	web depth
F_y	yield stress
h	web clear depth
k_v	shear buckling coefficient
l, L	length of the specimen
M_p	plastic moment
t	plate thickness
t_f	flange thickness
t_w	web thickness
V_{crl}	elastic shear buckling load $(=\tau_{crl} \times A_w)$
V_n	shear strength
V_y	shear yield strength $(= \tau_y \times A_w)$
γ_{M1}	partial safety factor
λ_v, λ_w	non-dimensional slenderness $\left(= 0.76\sqrt{F_y/\tau_{crl}},\ \sqrt{V_y/V_{crl}}\right)$
τ_{cr}	shear buckling stress
τ_{max}	maximum shear stress
τ_y	shear yield stress

shear in cold-formed steel members was suggested in Pham and Hancock [33] and has been verified to reliably predict ultimate shear strength with respect to test and FE results for cold-formed steel beams.

This paper develops a design shear strength formula of the end web panels of plate girders and examines the application of the DSM to the end web panel of plate girders both experimentally and theoretically. For this, a series of shear tests was conducted using plate girders with a single double-sided transverse stiffener at supports, which were fabricated from 4.5-mm-thick steel plates with the nominal yield stress of 235 MPa and the ultimate stress of 400 MPa. Test results in literature [16,17] are compared with the shear strength formula for end web panels with two double-sided bearing stiffeners and interior web panels. The strength formula of the DSM for the shear strength of the web panel of plate girders undergoing shear buckling and post-buckling was calibrated to shear test results. Nonlinear analyses of tested sections were conducted to compare their results with test results. The design strength formula for the DSM was compared with existing specifications and test results. Design strength curves for the shear of thin web panels of plate girders were verified to be efficient for predicting the ultimate shear strength of end web panels of plate girders subjected to shear loads.

2. Test sections

2.1. Material properties

The structural steel grade was SS400 for the web and SM490 for the flange of plate girders fabricated for tests (KSD 3515) [34]. The minimum specified yield and ultimate tensile stresses were 235.0 MPa and 400.0 MPa, respectively, for SS400 and 315.0 MPa and 490.0 MPa, respectively, for SM490. Tensile coupon tests were conducted for pairs of flat coupons cut from the web and flanges of plate girders. All coupons were tested in a 250 kN UTM (Schimazu AUTOGRAPH AG 250kNG) at a displacement rate of 0.1 mm/min. The average yield and ultimate tensile stresses obtained from tensile coupon tests were 339.2 MPa and 441.1 MPa, respectively, for SS400 and 374.5 MPa and 549.2 MPa, respectively, for SM490. The average yield stress of the coupon cut from the web far exceeded the nominal yield stress by 44.3%. However, the tensile stress exceeded the nominal value by 9.3%. In addition, the yield and tensile strength of the coupon cut from flanges exceeded nominal strength by 19.9% and 12.2%, respectively. The measured elastic modulus was 2.0×10^5 MPa, and the average measured elongation was 36.5%. Stress–strain curves of SS400 and SM490 steel coupons are shown in Fig. 1(a) and (b).

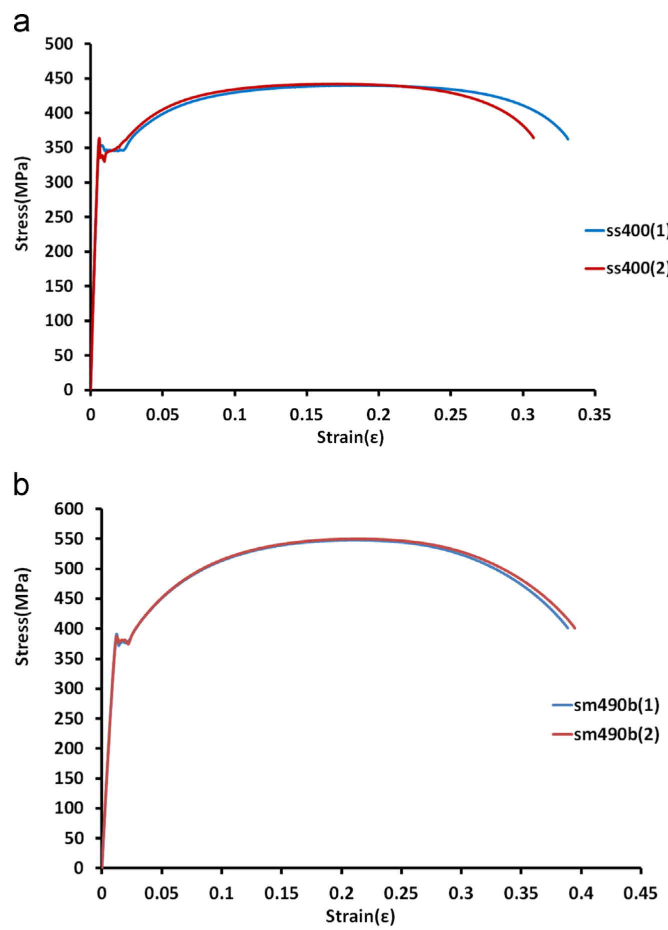

Fig. 1. Stress versus strain curves from coupon tests. (a) SS400 steel. (b) SM490 steel.

2.2. Section geometry

A series of pseudo-static shear tests was performed on the hybrid plate girders of a web 4.5 mm thick and flanges 12.0 mm thick whose nominal yield stresses were 235 MPa (SS400) and 315 MPa (SM490), respectively. The cross-sectional geometry of the tested plate girder is shown in Fig. 2. The plate girders were fabricated by continuous fillet welding along flange-to-web joints. The size of the fillet weld was determined as 4.5 mm according to AISC specifications [24]. A single double-sided transverse stiffener of 12 mm in thickness was attached to the both sides of the web at