Journal of Constructional Steel Research 138 (2017) 513–525

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

Shear strength of tapered end web panels

Mohammed H. Serror *, Basem H. Abdelbaset, Hesham S. Sayed

Department of structural engineering, Faculty of Engineering, Cairo University, Egypt

ARTICLE INFO

Article history:
Received 27 October 2016
Received in revised form 26 July 2017
Accepted 7 August 2017
Available online xxxx

Keywords:
Numerical study
Tapered end web panel
Elastic shear buckling
Nominal shear strength
Diagonal tension action

ABSTRACT

Hot rolled beams can show insufficient strength or inertia and result in the utilization of steel plate girders in the design. For cost-effective design, tapered plate girders are employed, where the depth of the end web panel is linearly varied with the panel length. In most of design codes, the shear capacity is well estimated for prismatic web panels, with reasonable accuracy. The tapered web panels, however, are lacking investigation. The objective of this numerical study is to examine the effect of different geometric parameters of tapered end web panel on the elastic shear buckling and the nominal shear strength. The geometric parameters in question are, namely: tapering angle; tapering direction; panel aspect ratio; web slenderness ratio; flange to web thickness ratio; and attachment of transversal vertical stiffeners to panel ends. The finite element method has been employed, where linear elastic buckling and nonlinear inelastic post-buckling analyses have been performed. The numerical results have been verified against classical web buckling theory, design codes, and experimental results published in the literature. Furthermore, regression analysis has been performed for the obtained results, where new design rules have been proposed for both elastic and nominal shear strength. It has been reported that tapered end web panels possess post-buckling strength that is highly dependent on the geometric parameters.

1. Introduction

In plate girders with practical spans, the induced shear force in the web is relatively low as compared with the axial forces in the flanges, as resulting from flexure. Consequently, the thickness of the web plate is generally much smaller than that of the flanges; and in turn, the web plate buckles at a relatively low value of shear load. The slender plate girder webs, however, do not fail in elastic buckling; unlike most post-buckling theories, they exhibit significant post-buckling strength. This is attributed to the fact that the girder behaves as a truss with top and bottom cords (the flanges) and verticals (the transverse stiffeners) where the equilibrium is achieved through developing diagonal tension action. Hence, the attachment of transverse stiffeners to the web plate allows for post-buckling shear strength. Accordingly, the nominal shear load of the web panel (V_n) can be computed by adding the elastic buckling load (V_{cr}) and the post-buckling load (V_{pb}) [1]. Timoshenko and Gere [2] established a closed-form solution for V_{cr} as given in Eq. (1). The web panel was assumed to be under simply supported boundary conditions, where E is the modulus of elasticity; ν is Poisson's ratio; d is the web plate depth; t_w is the web plate thickness; and K is the shear buckling coefficient that depends on web plate aspect ratio α as given in Eqs. (2) and (3).

$$V_{cr} = d\, t_w\, K \frac{\pi^2 E}{12\,(1-\nu^2)\,(d/t_w)^2} \quad (1)$$

$$K = 4 + \frac{5.34}{\alpha^2} \qquad \text{for } \alpha < 1 \quad (2)$$

$$K = 5.34 + \frac{4}{\alpha^2} \qquad \text{for } \alpha \ge 1 \quad (3)$$

In plate girder, however, the web panel is partially restrained against rotation at the flange-web juncture. Experimental and numerical research works have been conducted to obtain V_{cr} at different boundary conditions and geometric characteristics of the web panel [3–11]. Lee and Yoo [8] proposed formulas to calculate K considering the flange restrain to web rotation, as given in Eqs. (4)–(7). K_{sf} is the K coefficient in case the flange is fully restraining web rotation (no-rotation support); K_{ss} is the K coefficient in case the flange is not restraining web rotation (free-rotation support) as per Eqs. (2) and (3); and t_f is the flange thickness. As reported in Eq. (7), if t_f/t_w ratio exceeded the value of two, the web plate can be considered as partially restrained against rotation. This case of partial restraint is attributed in this study as 80% fixation to the web.

$$K_{sf} = \frac{5.34}{\alpha^2} + \frac{2.31}{\alpha} + 8.39\alpha - 3.44 \qquad \text{for } \alpha < 1 \quad (4)$$

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

$$K = 0.8\,(K_{sf} - K_{ss})(1 - 0.67(2 - t_f/t_w)) \qquad \text{for } 0.5 \le t_f/t_w < 2 \quad (6)$$

$$K = K_{ss} + 0.8\,(K_{sf} - K_{ss}) \qquad \text{for } t_f/t_w \ge 2 \quad (7)$$

* Corresponding author.
E-mail address: serror@eng.cu.edu.eg (M.H. Serror).

http://dx.doi.org/10.1016/j.jcsr.2017.08.012
0143-974X/© 2017 Published by Elsevier Ltd.

Tapered web panels have been investigated through numerical and experimental models that have been established to evaluate the shear strength [12–19]. Furthermore, modification to equation of Timoshenko and Gere [2] has been elaborated [20–23]. Mirambell and Zarate [20] proposed formulas to calculate V_{cr} of tapered plate girder web based on Timoshenko and Gere [2] formula. Eqs. (8) and (9) report the adopted K coefficient, where c_1 to c_4 are factors depend on the bottom flange inclination angle β; η is the flange width to larger beam depth ratio, λ_f is the flange slenderness ratio; and α is the web aspect ratio.

$$K = (c_1\eta^{c_2} - c_3\eta^{-c_4}\lambda_f - 4) + 4/\alpha^2 - 5(\beta)^{0.8}(\alpha - 1) \qquad \alpha \geq 1 \qquad (8)$$

$$K = (c_1\eta^{c_2} - c_3\eta^{-c_4}\lambda_f - 5.34) + 5.34/\alpha^{1.8} + 2(\beta)^{0.8}(\alpha - 1) \qquad \alpha < 1 \qquad (9)$$

In addition, Mirambell and Zarate [20] proposed formula to calculate the nominal shear strength V_n as reported in Eq. (10), where $??_{cr}$ is the critical shear stress; h_o is the smallest web depth; σ_{bb} is the strength of the diagonal tension; g is the tension field width; and ϕ is the angle of tension field inclination.

$$V_n = \tau_{cr} h_o t_w + \sigma_{bb} g t_w \sin(\phi) \qquad (10)$$

Bedynek [23] studied four different typologies with respect to direction of tension filed action and stress state in the inclined flanges. Typologies 1 and 3 represent tension field in short direction, with compression and tension stress states in the inclined flange, respectively. Typologies 2 and 4 represent tension field in long direction, with tension and compression stress states in the inclined flange, respectively. Hence, four formulas for the K coefficient, to calculate V_{cr}, were proposed as reported in Eqs. (11)–(14), where α is the plate aspect ratio and β is the flange inclination angle.

$$K_{Typology-1} = 5.5\alpha^{0.8}\tan(\beta) + 8.7\alpha^{-0.4} \qquad (11)$$

$$K_{Typology-2} = 10.6\alpha^{0.5}\tan(\beta) + 8.0\alpha^{-0.4} \qquad (12)$$

$$K_{Typology-3} = 47.0\alpha^{1.8}\tan^2(\beta) + \left(0.3 + 3.7\alpha - 0.5\alpha^2\right)\tan(\beta) + \left(0.5\alpha^2 - 4.2\alpha + 13\right) \qquad (13)$$

$$K_{Typology-4} = 62.0\alpha^{1.64}\tan^2(\beta) + \left(4.6\alpha - 0.7\alpha^2 - 2.8\right)\tan(\beta) + \left(0.44\alpha^2 - 3.4\alpha + 12\right) \qquad (14)$$

Furthermore, Bedynek [23] proposed formulas to calculate the nominal shear strength V_n as reported in Eqs. (15) and (16). V_{Web} and $V_{Flanges}$ are the shear contribution of panel web and flanges, respectively. V_{Resal} is an additional vertical component derived from the axial force in the inclined flange. This phenomenon is called Resal effect and can be favorable or not. For typologies 1 and 2 where the moment of inertia of the cross-section increases with the increase of internal forces, the vertical component acts against shear force and reduces it, thus the ultimate shear resistance is greater (positive influence). For typologies 3 and 4 the opposite situation was observed. It is worth noting that detailed equations for V_{Web}, $V_{Flanges}$ and V_{Resal} are given in the reference [23].

$$V_{n\_Topologies\_1\&2} = V_{Web} + V_{Flanges} + V_{Resal} \qquad (15)$$

$$V_{n\_Topologies\_3\&4} = V_{Web} + V_{Flanges} - V_{Resal} \qquad (16)$$

Basler [3] reported the post-buckling shear strength, where the web buckles in waves or wrinkles running in the tension field direction while the compressive stresses continue to increase. It was assumed that the diagonal tension does not develop near the flange-web juncture; instead, it develops near the transverse stiffeners, since the flanges do not possess sufficient flexural rigidity around its weak axis. The contribution of transverse stiffeners to the nominal shear strength has been investigated [11], where the minimum area and inertia have been reported to provide sufficient restraint to the web and to develop the diagonal tension action. Most of the numerical models were based on Basler [3] theory assuming that for tapered web panel the critical shear load may be evaluated according to the classical theory for simply supported rectangular plates, however, using the average depth of the trapezoidal panel. Porter et al. [6] developed a sway failure mechanism for the web panel based on the assumption that the flanges are able to anchor the diagonal tension. The mechanism supported the web deformations as exhibited in experimental reports, and revealed that the web reaches failure as soon as the plastic hinges are developed at the flanges. Thus, it is necessary to calculate the shear capacity derived from the tapered web and the influence of the flange bending resistance on the shear capacity. Lee and Yoo [8,10] developed a formula to calculate the nominal shear strength of prismatic plate girder as the superposition of critical buckling shear strength and post-buckling shear strength. Modification factors have been proposed to account for strength reduction due to high slenderness ratio and large initial imperfection.

The design codes [24–27] follow the formula proposed by Timoshenko and Gere [2] to calculate the elastic shear buckling load for prismatic plate girders, where simply supported boundary conditions are assumed. The Eurocode EC3 [24] employed the equations established by Timoshenko and Gere [2] to calculate V_{cr}, as reported in Eqs. (1)–(3). Similarly, AISC, AASHTO and Egyptian codes [25–27] employed Timoshenko and Gere [2] equations to calculate V_{cr}, however, using an approximate formula for the K coefficient as:

$$K = 5 + 5/\alpha^2 \qquad (17)$$

On the other hand, the Eurocode EC3 [24] adopts the model of Porter et al. [6] to calculate the nominal shear strength for prismatic plate girders; meanwhile, the American and Egyptian codes [25–27] adopt the model of Basler [3]. For tapered plate girders, the Eurocode EC3 [24] allows to use the same rules of prismatic plate girders providing that the angle of flange inclination is < 10 degrees. If the angle is exceeding 10 degrees, the same rule may be applied using the larger panel depth.

In this study, FE analysis has been performed to investigate the effect of different geometric parameters on the elastic and nominal shear strength of tapered end web panel. The parameters in question are: tapering angle; tapering direction; panel aspect ratio; web slenderness ratio; flange to web thickness ratio; and attachment of transversal vertical stiffeners to panel ends. The numerical results have been verified against classical web buckling theory and experimental results published in the literature. Furthermore, regression analysis has been performed for the obtained results, where new design rules have been proposed for both elastic and nominal shear strength of tapered end web panels.

2. Numerical model

Fig. 1 illustrates four structural models that have been established and examined for two different directions of the diagonal tension action (short and long directions) with and without transverse stiffeners. Consequently, four configurations have been considered in this study, namely: CONFIG-A for short diagonal tension without stiffeners; CONFIG-B for short diagonal tension with stiffeners; CONFIG-C for long diagonal tension without stiffeners; and CONFIG-D for long diagonal tension with stiffeners. The boundary conditions adopted for the tapered end web panel would match closely the behavior of a simply supported beam under loading at mid-span. The beam shown in Fig. 1