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Failure loads of web panels loaded in pure shear

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ABSTRACT

The effect of tension field action on the ultimate shear strength of web panels has been considered using different approaches. These approaches lead to wide range of predicted shear capacities especially for slender web panels that have large web width–thickness ratios. Therefore, this study aims to investigate the actual behavior of web panels loaded in pure shear. Nonlinear analysis has been conducted using three-dimensional finite element model of plate girder web panels. The study covers a wide range of web width–thickness ratios, web-panel aspect ratios as well as flange bending stiffness. Results show that both short as well as long web panels have substantial post local buckling capacities. However, this additional capacity decreases by increasing the web panel aspect ratios. Moreover, the diagonal tension field is often formed off the web panel diagonal. In addition there is no remarkable effect of the flange bending stiffness on the ultimate shear strength. Based on the numerical results an empirical equation is proposed. Finally comparison has been done with AAHSTO, AISC, and Eurocode-3 as well as different theories.

1. Introduction

Web elements in plate girders resist primarily the induced shear stresses by the classical beam action in which equal tensile and compressive diagonal stresses are developed. If the web is stocky enough, failure will occur by shear yielding. However, the practical design of plate girders yields to thin webs and consequently shear buckling waves are happened. Once the web plate buckles, it loses its capacity to carry any additional diagonal compressive stresses, and the web carrying the additional shear loads by the tension field action. Several approaches evaluate the post local shear buckling capacity. They are varied in the assumptions that whether the flanges are flexible so the diagonal tension field does not developed near the flange web junction, or the flanges are stiff so they are able to anchor the diagonal tension field. Also, these approaches have different definitions for the inclination and the variation of stresses across the diagonal tension field.

Several researchers presented analytical and experimental investigations for the strength of plate girders in shear. Basler [1] derived analytically an equation that determines the ultimate shear capacity of plate girders loaded in pure shear. His equation accounts for the tension field that happened after buckling of the web. He assumed that the flanges are very flexible so they cannot withstand the lateral loads that come from the tension field. K.C.Rockey, et al. [2] conducted experiments on different girder models showing the effect of the flange stiffness on the ultimate shear capacity. They postulated a collapse mechanism in which plastic hinges are formed in the flanges. They assumed the inclination of the diagonal tension field equal to that of the geometric web panel diagonal line. Chern and Ostapenko [3] assumed a collapse mechanism similar to that of Basler [1] except they allow the variation of the membrane stress across the whole web section. They stated that the flanges would contribute to the strength of the girder and they allowed for the development of a frame type mechanism in which hinges are formed in the flanges over the transverse stiffeners. D.M.Porter [4] postulated that a collapse mechanism allows for the presence of plastic hinges in the flanges and the variation of the tension field band inclination with the rigidity of the flanges. Torosten Hoguland [5] presented the rotated stress field method. He found that this method gives best agreement with 366 tests on steel plate girders as well as 93 tests on aluminum alloy plate girders in shear. Sung C. Lee, et al [6,7] showed that flanges have little effect on the development of the post buckling strength of web panels, and the shear resistance capabilities of the flanges are the main cause of the little difference in the shear capacities of beams and not the flexural rigidity of the flanges. They also developed method for predicting the ultimate shear strength of web panels with large aspect ratios. M.M. Alinia et al. [8] showed numerically that the state of stresses on the two faces of a square web plate are different due to the secondary bending stresses induced by large out of plane deformations, and both the tensile and compressive principal stresses increase after buckling near the edges of the web plates. However beams are often subject to combined bending and shear. Therefore a lot of researchers studied the interaction effect of shear force and bending moments. Basler [9] drew an interaction diagram between bending and shears strength of plate girder. He assumed that the shear strength of the section will not be affected by the presence of the applied moments unless they are higher than that carried

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by the flanges only. Hoglund [5] listed an interaction equation similar to that presented by Basler [9] but it allows for the reduction in the shear resistance carried by the flanges when the applied moments are lower than that carried by the flanges. Roberts and Shahabian [10] conducted experiments on series of I-section girders to determine the ultimate resistance of slender web panels to combinations of bending, shear, and patch loading. M.El Aghoury and M.T. Hanna [11] studied numerically the interaction behavior of slender I-sections web panels that are subjected to combined bending and shear forces. They found that the web panel aspect ratios will not significantly affect the combined ultimate shear bending strength as well as the post local buckling strength gained by the section. They also proposed an empirical interaction equation.

The main aim of this work is to investigate the structural behavior of web panels loaded in pure shear as depicted in Fig. 1. The web panels are provided with two vertical end plates. A group of built up I-sections with different flange and web width–thickness ratios as listed in Table 1 are studied. It is assumed that the cross sections are made of mild steel with a yield stress of 240 MPa. The web width–thickness ratios, H_w/t_w, range from 25 to 300. The flange width–thickness ratios, b_f/t_f, range from 5 to 20. In addition, for each cross section, the web panel aspect ratios, $a = S/H_w$, range from 1 to 6.

Table 1
Sections studied.

Sec. no.	H_w (mm)	t_w (mm)	b_f (mm)	t_f (mm)	H_w/t_w	b_f/t_f
S1	500	20	100	20	25	5
S2	500	10	100	20	50	5
S3	1000	10	100	20	100	5
S4	1000	6.6	100	20	150	5
S5	1000	5	100	20	200	5
S6	1500	6	100	20	250	5
S7	1500	5	100	20	300	5
S8	500	20	200	20	25	10
S9	500	10	200	20	50	10
S10	1000	10	200	20	100	10
S11	1000	6.6	200	20	150	10
S12	1000	5	200	20	200	10
S13	1500	6	200	20	250	10
S14	1500	5	200	20	300	10
S15	500	20	300	15	25	20
S16	500	10	300	15	50	20
S17	1000	10	300	15	100	20
S18	1000	6.6	300	15	150	20
S19	1000	5	300	15	200	20
S20	1500	6	300	15	250	20
S21	1500	5	300	15	300	20

2. Finite element model

2.1. Description of the model

A non-linear finite element model is developed by using "COSMOS/M" finite element package [12]. Four node isoparametric shell element is used in the model. This element allows for both geometric and material non-linearities. The mesh density has been chosen to make the element aspect ratio on average equal to 1. Fig. 2 shows an example of the model that is used in the study. The boundary conditions are applied so that the shear deformations in the plain YZ are allowed. This is done as follows: translation in X directions isprevented for all nodes on the four sides SV1, SV2, SH1, and SH2. In-addition, nodes on side SV1 are prevented from translation along Z-directions, while nodes on side SV2 are prevented from translations along Y and Z directions. The material behavior is assumed to be elastic–perfect plastic stress strain curve obeying von Mises yield criterion. Vertical load acting downward is applied on side SV1; hence, vertical upward reaction is generated on side SV2. To eliminate the bending moments developed and produce case of pure shear loading two equal and opposite horizontal load, $F = V. S/H_w$, are applied on sides SH1, and SH2. The loads applied are illustrated in Fig. 1. Note, two vertical end plates are added at the two ends of the model (SV1, SV2). Load control technique is used to control the increment of the external loads. The Newton–Raphson iterative technique and the tangential stiffness matrix are implemented to solve the set of nonlinear equations at each load increment. Failure is defined as the load at which the iteration is not converged, where the slope of the tangential stiffness matrix becomes nearly zero.

2.2. Effect of initial imperfections

Steel structural elements usually contain geometric imperfections as well as residual stresses that arise from manufacturing and handling. However, Lee et al. [7] mentioned that the ultimate shear stresses are not affected by the residual stresses; hence, in this work the geometric imperfections will only be considered. Generally, geometric imperfections are classified into global and local imperfections. First, global imperfections represent the geometric defects that may be present in the member length such as bending, twisting, etc. However, local imperfections represent the changes in the shape of the steel plates comprising the cross section from its ideal geometry.

In this study, the assumed local imperfection pattern consists of one half sine wave in the transverse direction (Y-axis), and series of half sine waves in the longitudinal direction (Z-axis) as shown in Fig. 3-a. The number of the longitudinal half sine waves is assumed equal to the web panel aspect ratio, $a = S/H_w$. However, the overall geometric imperfections are considered by modeling the member with one half sine wave along its whole length, S, as shown in Fig. 3-b. The maximum amplitude at the member mid-length is the overall imperfection value.

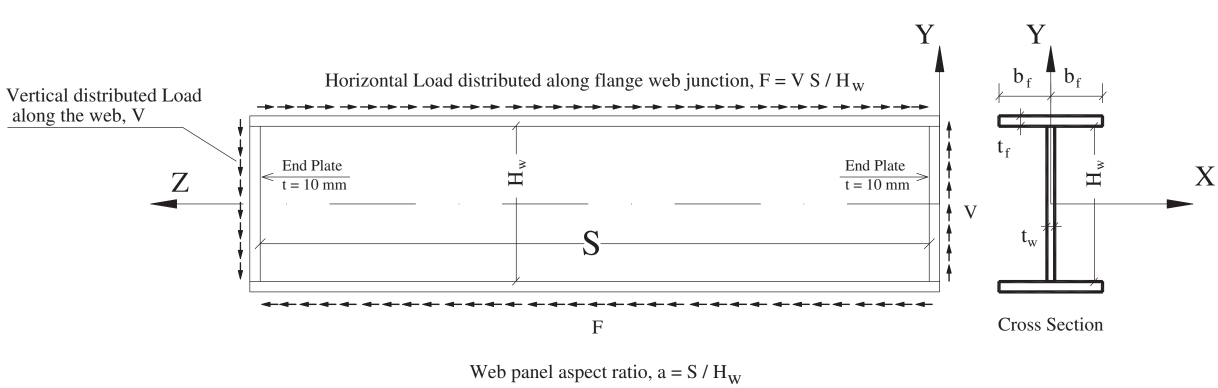

Fig. 1. Case of study.