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Shear failure characteristics of steel plate girders

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ABSTRACT

A number of full-scale plate girders are modeled and analyzed to determine their shear failure mechanism characteristics. An objective of this numerical nonlinear large deflection elastoplastic finite element study is to clarify how, when, and why plastic hinges that emerge in experimental tests actually form. It is observed that shear-induced plastic hinges only develop in the end panels. These hinges are caused by the shear deformations near supports and not due to bending stresses arising from tension fields. Also, a comparison between the ultimate capacity of various plate girders and different codes and theories is presented. Finally, it is shown that simple shear panels, in the form of detached plates, do not accurately represent the failure mechanism of web plates.

1. Introduction

Plate girders are designed to support heavy loads over long spans such as building floors, bridges and cranes; where standard rolled sections or compound girders are not answerable. Modern plate girders are, in general, fabricated by welding together two flanges, a web and a series of transverse stiffeners. Flanges resist applied moment, while web plates maintain the relative distance between flanges and resist shear. In most practical ranges, the induced shearing force is relatively lower than the normal flange forces. Therefore, to obtain a high strength to weight ratio, it is common to choose deep girders. This entails a deep web whose weight is minimized by reducing its thickness. Various forms of instabilities, such as shear buckling of web plates, lateral-torsional buckling of girders, compression buckling of webs, flange-induced buckling of webs, and local buckling and crippling of webs are considered in design procedures.

Due to the slenderness of web plates, they buckle at early stages of loading. Therefore, one important design aspect of plate girders is the shear buckling and failure of web elements. Webs are often reinforced with transverse and in some cases with longitudinal stiffeners [1–3] to increase their buckling strength. A proper web design involves finding a combination of optimum plate thickness and stiffener spacing that renders economy in terms of material and fabrication cost. The design process of plate girder webs are commonly carried out within two categories: (i) allowable stress design based on elastic buckling as a limiting condition; and (ii) strength design based on ultimate strength, including postbuckling as a limit state. Till 1960s, the elastic buckling concept was basically used in the design of plate girders and the postbuckling strength was only indirectly accounted for by means of lowering safety factors.

Wilson [4] first discovered the postbuckling behavior in 1886, and Wagner [5] developed the theory of uniform diagonal tension for aircraft structures with very thin panels and rigid flanges in 1931. In late 1950s, Basler and Thurliman [6] took a different approach and carried out extensive studies on the postbuckling behavior of plate girder web panels. They assumed that tension field develops only in parts of the web and that flanges are too flexible to support normal stresses induced by the inclined tension field. In other words, yield zones form away from flanges and merely transverse stiffeners act as anchors. Their alleged assumption was in contrast to the Wagner's [5]; but later other researchers like Fujji [7] showed that the Basler's formula was given for complete tension field instead of limited band. Further research works by Basler [8–10] paved the way for the American Institute of Steel Construction (AISC) [11] and the American Association of Steel Highway and Transportation Officials (AASHTO) [12] to adopt the postbuckling strength of plates into their specifications. By moving towards applying the limit state design concept in the design of steel structures, SSRC [13] introduced a number of modified failure concepts to achieve a better correlation between theories and test results.

On the other side, the Cardiff model developed by Porter et al. [14] was adopted into the British Standards [15]. They also assumed that inclined tension fields only develop in a limited portion, but that flanges do contribute to the postbuckling strength by absorbing normal stresses from tension fields; and that as a result, girders collapse when plastic hinges form in their flanges.

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Notations

A	area of end-post/stiffener
a	panel width
b_f	flange width
c	position of flange plastic hinge
E	elastic modulus
e	width of end stiffener
f_y	material yield stress
h_w	web height
k	shear buckling coefficient
L	girder span
t_f	flange thickness
t_s	thickness of intermediate stiffeners
t_{se}	thickness of end stiffeners
t_w	web plate thickness
Δ	in-plane deflection of girder
δ	out-of-plane displacement of web panels
υ	Poisson's ratio
σ_x, σ_y	normal stresses
τ_{cr}	critical shear stress
τ_{xy}	shear stress

Basler [10], Porter et al. [14], Takeuchi [16] and Herzog [17] assumed that the diagonal tension field develops in a limited portion of the web. In contrast, Fujji [18], Komatsu [19], Chern and Ostapenko [20] and Sharp and Clark [21] assumed that diagonal tension spreads all over the panel, but with different intensity. The Steinhardt and Schroter's [22] assumption, lies half way between the two previous assumptions. Hoglund [23–25] developed a theory for transversely stiffened and unstiffened plate girders. He used the system of diagonal tension and compression bars to model web plates. His theory later became the basis for Eurocode 3 [26].

Although these classical failure theories assumed different yield zone patterns, the fundamental assumption that "compressive stresses that develop in the direction perpendicular to the tension diagonal do not increase any further once elastic buckling has taken place" was common in all of them. The application of this fundamental assumption to the whole web panel led to the well-known theory that the tension field action in plate girders with transverse stiffeners needs to be anchored by flanges and stiffeners in order for the webs to develop their full postbuckling strength.

Takeuchi [16] was the first to make an allowance for the effect of flange stiffness on the yield zone of web plates. Among the previous researchers, Fujji [18], Komatsu [19], Porter et al. [14] and Hoglund [25] assumed that the normal stresses induced by the tension fields are anchored by the top and bottom flanges and/or the combination of transverse stiffeners and adjacent panels. These normal stresses, thus, produce a beam mechanism in flanges and the ultimate capacity of plate girder is accompanied by the formation of plastic hinges in flanges. Their proposed theories, it seems, were invented to justify the formation of plastic hinges that had materialized in extensive experiments.

In other series of analytical and experimental works, Lee and Yoo [27–31] showed that flanges and transverse stiffeners do not necessarily behave as anchors. Their studies confirmed that intermediate transverse stiffeners are not subjected to compressive forces and that flanges are not subjected to lateral loadings. They further introduced an approach that was referred to as the shear cell analogy to resolve the discrepancy between their previous understandings and new findings. However, on reexamining, they noticed that the shear cell analogy does in fact contain a serious flaw. An important stress component was inadvertently omitted during the transformation process from a two-dimensional stress to an assembly of one-dimensional bar element.

Ever since Wagner [5] proposed the pre-mentioned fundamental assumption, no one has examined it critically. Although Marsh et al. [32] found that the diagonal compression at the tension corners of the web increased after buckling, they still concluded that flanges contribute to the shear capacity of panels due to their bending strength, which permits the development of some diagonal tension.

The assumed failure mechanisms in Basler, Cardiff and other mentioned models probably do not accurately represent the ultimate shear behavior of web panels, since they are significantly affected by bending stresses when panels undergo large postbuckling deformations and the pattern of yield zones at one face is different from the other [33]. In short, although the classical theories underestimate the buckling strength due to the negligence of torsional rigidity of boundary members, they give higher values for the ultimate shear strength, because of their overestimation in the postbuckling strength [31,34,35].

The nonlinear shear stress and normal stress interaction that takes place from the onset of elastic shear buckling to the ultimate strength state is so complex that any attempt to address this phenomenon using classical closed form solutions appear to be unsuccessful. The fact that there have been many theories for explaining this occurrence is evidence to the complexity of tension field action. The objective of this nonlinear large deflection elastoplastic finite element (FE) study is to clarify the mechanism of shear failure in steel plate girders; and to answer why, how, when, and where plastic hinges form. Other aspects of shear plate behaviors, such as their deformability and rigidity and strength degradation due to fatigue-induced cracks have previously been reported by the present first author and his colleagues [34–38].

2. Method of study

2.1. General

A detached web panel simulation model, a simply supported web plate in shear, or even single-panel experimental tests cannot truly represent the behavior of plate girder web plates, since:

(a) A web plate is bound to have some bending moments due to lateral loadings.
(b) The torsional rigidity of girder flanges must be accounted for in the rotational stiffness of panel boundary conditions. The true behavior of flange–web junction is neither simply supported nor clamped.
(c) In reality, flanges are allowed to move towards or apart from each other, and their weak axis second moment of area becomes an important factor in this regard. A free or restrained in-plane movement of panel edges cannot represent the real behavior of web plates.
(d) The number of sub-plates created by intermediate transverse stiffeners and conditions of end-posts (end stiffeners) have considerable effects on the behavior of plate girders.

Therefore, in order to investigate the explicit shear failure mechanism of plate girders, complete girders with appropriate boundary restraints must be simulated. In this research, simple