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Shear buckling and stress distribution in trapezoidal web corrugated steel beams

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ABSTRACT

Due to their lightweight and superior load carrying capacity, corrugated web steel beams (CWSBs) have gained popularity in the last few decades. CWSBs are known to fail at much higher loads compared to stiffened flat web beams. To understand their shear response, a series of three-point load tests were performed on five shear-critical trapezoidal corrugated web beams. The test results confirmed the existence of the three shear buckling modes of failure: local, global, and interactive. In addition, all tested beams were observed to have a residual strength that is about half of their ultimate load carrying capacity regardless of the shear buckling mode. Results of the nonlinear finite element analysis showed that the shear stress is at its maximum and uniformly distributed throughout the web until buckling, afterwards, it decreases and its distribution is uneven while the entire resistance is provided by the increased tensile stress. Furthermore, stocky corrugated webs were shown to reach shear yield strength. Comparison between existing analytical models for the estimation of shear strength against test data showed that EN-1993-1-5 is accurate and conservative enough for an economic design.

1. Introduction

To efficiently use the full strength of the material, the web, in beams or girders, should be very thin [1]. However, such thickness makes the manufacturing process very difficult. This issue has been solved by utilizing corrugated webs instead of very thin plates. Corrugated Web Steel Beams (CWSBs) are fabricated by welding corrugated steel webs between two flanges (Fig. 1).

A CWSB is a structural element that differs from traditional hot rolled I-beams in many aspects. The corrugated nature of the web provides an enhanced strength against shear buckling failure. Laboratory tests conducted by Li *et al.* [2] demonstrated that CWSBs are 1.5–2 times more resistant in shear than flat web beams. Generally, the webs are thin corrugated steel plates with thickness ranging from 1 mm to several millimeters. Thus, CWSBs are considered as an economical solution compared to hot profiled beams due to the weight reduction. For instance, Hamada *et al.* [3] found that corrugated web girders are 9–13% lighter than stiffened girders with flat webs.

The corrugated nature of the webs is assumed to enhance their shear strength, however, the corrugations contribute to their ultimate failure due to the "*accordion effect*" [4,5]. Failure of CWSBs may be due to steel yielding or buckling of the web plate. Shear buckling of corrugated webs may take three forms: local, global, or interactive shear buckling, depending on the geometry characteristics of the steel web plates. The interactive buckling occurs due to the interaction between global and local buckling and is influenced by the geometry of corrugated webs [6].

Stiffened plate girders are known to have a strength that is much larger than their buckling loads [7]. Until 1960, design of plate girders was based on buckling. However, acknowledging the fact that the girders can carry loads after buckling led to the development of new theories. These theories were adopted in many design specifications by incorporating the post-buckling strength of plate girders. Post-buckling of plate girders has been the subject of extensive research since Wilson's [8] observations on the residual load bearing capacity of bridge plate girders. Wagner [9] and Kuhn [10] developed the concept of uniform, complete, diagonal tension theory of webs subjected to shear loading (Fig. 2). Basler and Thurlimann [11] developed the limited tension field theory for the post-buckling of webs in shear. Cardiff model [12], which was adopted by the British Standard, considers a diagonal tension band with failure occurring when the yield spreads all over the band and hinges form in the flanges. Eurocode 3 EN1993-1-5 [13] adopted Höglund's model [14], which is an extension to an earlier theory that is based on the rotated stress field method. Experimental results showed that the model that considers plate girders with transverse stiffeners gives conservative shear strength results for slender web panels. A critical review of the above models was reported by [7].

Post-buckling strength is achieved when the web's diagonal devel-

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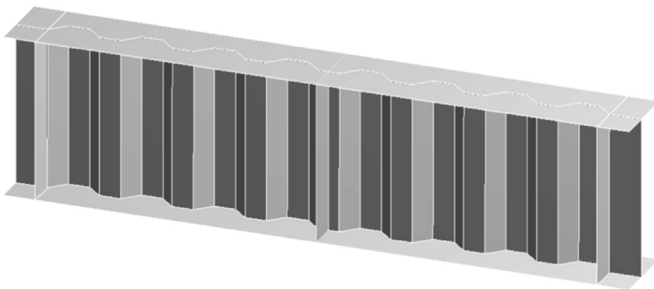

Fig. 1. Beam with trapezoidal corrugated web.

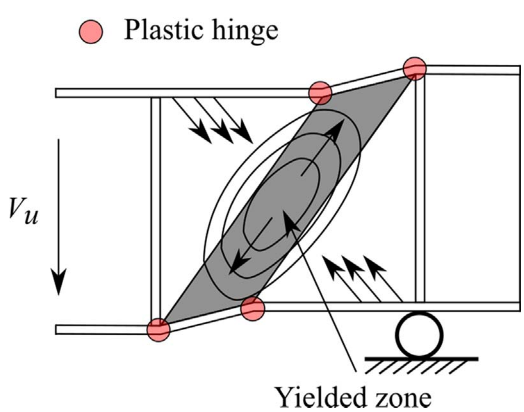

Fig. 2. Tension-field action and frame mechanism.

ops a tension field (tension-field action). Prior to that, the shear stress distribution is uniform all over the web panel [7]. The tension-field action is affected by the contribution of the flexural stiffness of the flanges.

With reference to Fig. 2, failure of the web panel occurs as a result of the formation of frame mechanism, accompanied by the development of plastic hinges in the flanges [15] and yielding of the panel zone under tension.

The aims of this research are to study experimentally and numerically the shear response of trapezoidal corrugated webs from the pre-buckling stage until ultimate failure, to review existing analytical models for the estimation of shear strength of corrugated webs and verify their performance when compared to the new test results.

2. Review of previous related studies

In this section, an overview of some of the reported experimental and numerical analysis data on CWSBs is presented. Previously published analytical models for the estimation of the shear strength of CWSBs are presented and briefly reviewed. The analytical models discussed in this section include the models based on the assumption of interaction between local and global buckling [6,16–18], in addition to the model by the European standard EN 1993-1-5 [13].

2.1. Experimental and numerical studies

Many studies dealt with the shear response of trapezoidal corrugated webs, either experimentally or numerically. For instance, Abbas *et al.* [19] reviewed several tests conducted in the United States and Europe. Elgaaly *et al.* [20] reported four tests by Smith [21] and 42 tests by Hamilton [22] and carried out nonlinear finite element analysis (NLFEA) to depict the behavior of tested specimens. All tested specimens were shown to fail due to shear buckling. Comparison between numerical and test results was satisfactory, despite the presence of initial imperfections in the tested specimens. Later, Elgaaly and Seshadri [23] employed NLFEA using Abaqus® to depict the behavior of girders with trapezoidal corrugated webs when subjected to the three loading conditions: shear, uniform bending, and local discrete compressive loads on the top flange. The authors introduced the initial imperfection in the form of a double sine wave with varying magnitudes and an increasing number of finite elements across the corrugation fold width and found that introducing the imperfection resulted in a good estimation of the experimental shear capacity, especially when three or four elements are used to mesh the corrugation folds.

Gil *et al.* [24] carried out laboratory tests and NLFEA considering the effects of both, material and geometric nonlinearities, and proposed a new shear buckling formula applicable to all three types of shear buckling modes. They reported that results of the NLFEA performed using Abaqus® software were in a relatively good agreement with test results, with a maximum error of 22%.

To propose an alternative shear strength design criteria suitable for trapezoidal corrugated webs, Moon *et al.* [4] performed three laboratory tests on 6-meter long girders and found that one of the specimens failed following the initial imperfection contour lines, while the other two failed due to global and interactive shear buckling modes. In addition, the authors confirmed that the shear strain is uniform across the web panels, thus proving the pure shear stress state until the buckling stage, after which the shear stress dramatically decreased.

More recently, Nie *et al.* [25] tested eight steel girders with trapezoidal corrugated webs, then carried out an extensive parametric study based on a series of linear elastic buckling analysis. The authors then proposed five formulas for the estimation of shear strength for five different initial geometric imperfections.

Hassanein and Kharoob [26] employed NLFEA to check the validity of existing analytical models in approximating the shear strength of trapezoidal corrugated webs. They introduced the initial imperfection, which is inherently present in all manufactured CWSBs, using the 1st eigenmode shape with a magnitude equal to the web thickness. The authors concluded that their model, which adopted the original model by Sause and Braxtan [18], was the best among all existing models, however, they recommended checking this finding through laboratory tests.

2.2. Existing analytical models

This section presents Eurocode 3 EN 1993-1-5 design method and four of the recent analytical prediction models for the estimation of shear strength of trapezoidal corrugated webs. The analytical models include those developed by El-Metwally [16], Driver *et al.* [17], Yi *et al.* [6], and Sause and Braxtan [18]. The presentation is made considering the notations in Fig. 3.

2.2.1. Eurocode 3 (EN 1993-1-5)

The development of this model was based on an earlier work by Höglund [14]. In this model, the critical shear buckling, V_{EC3}, is estimated using Eq. (1):

$$V_{EC3} = \rho_{EC3}\tau_y h_w t_w \tag{1}$$

in which τ_y is the material shear yield stress, which is determined assuming the Von Mises yield criterion as $\tau_y = f_{yw}/\sqrt{3}$, where f_{yw} is the uniaxial yield stress of the web's material. ρ_{EC3} is a reduction factor (i.e., normalized shear strength) for shear buckling according to Annex D of EN 1993-1-5 [13], it is the lesser of the values of reduction factors $\rho_{EC3,L}$ and $\rho_{EC3,G}$ defined for local and global shear buckling, respectively.

For local shear buckling, the reduction factor $\rho_{EC3,L}$ is defined as:

$$\rho_{EC3,L} = \frac{1.15}{0.9+\lambda_L} \quad \leq 1.0 \tag{2}$$

where λ_L is the slenderness ratio:

$$\lambda_L = \sqrt{\frac{\tau_y}{\tau_{cr,L}}} \tag{3}$$