



Tapered plate girders under shear: Tests and numerical research

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ABSTRACT

This paper presents an experimental and numerical research on tapered steel plate girders subjected to shear. Experimental tests included four small-scale tapered steel plate girders. Research was focused on both, critical shear load and ultimate shear resistance. Moreover, the post-buckling behaviour of tapered plates was studied.

Further, some parametric studies with various geometries of tapered panels were done in order to find the most favourable design situations. The analysed parameters were: the panel aspect ratio, the inclined flange angle, the web and the flange slenderness.

Numerical simulations allowed distinguishing four different typologies of tapered plate girders which should be considered separately in design because of their different behaviour. Verification of the simplified procedure for tapered plates proposed in Eurocode EN 1993-1-5 allowed concluding that for some cases the estimation of the ultimate shear resistance is situated on the unsafe side and need to be revised.

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1. Introduction

In building and civil engineering structures with large dimensions and subjected to high loads welded plate girders are currently designed. For efficient design it is usual to choose a relatively deep and non-prismatic girder. In these cases the web may be quite slender and so it may be prone to shear buckling. Although the shear buckling phenomenon has been widely studied for prismatic plates, there are very few theoretical and experimental investigations into the structural response of tapered steel plate girders under increasing shear load up to failure. Just as Galambos [1] points out, more work is required to develop general design procedures for the ultimate strength of steel panels with variable depth. There are no rules in current steel codes for the design of tapered plate girders.

The current design code for plated structural elements, EN 1993-1-5 [2] proposes to determine the ultimate shear resistance of tapered plate girders as prismatic ones. In order to evaluate this simplification, some experimental tests and a wide parametric study with different geometries of tapered plate girders were conducted.

This paper is divided into three principal parts. First of all the experimental tests on four tapered steel plate girders is presented. The experimental results were also used to validate a numerical FE model. Next part of this work is focused on the parametric study

and research on the influence of geometric parameters and structural imperfections (residual stress) on the ultimate shear strength. A lot of numerical simulations with different geometric parameters, as the slope of the flange and the aspect ratio, were done. Finally, the outcome of these simulations was compared with the results obtained according to EN 1993-1-5 and discussed.

2. Shear models for tapered plate girders

2.1. State of art

The behaviour of the rectangular steel plates subjected to shear load was deeply studied during last century and different theories were developed in order to describe and analyse the mechanisms that take place during the post-buckling state and finally, to determine their ultimate shear capacity. Some of them are implemented in design codes: the Rotated Stress Field Model developed by Höglund [3,4] and the Tension Field Model developed in Cardiff and Prague by Porter et al. [5] and Rockey and Škaloud [6].

However, these models are based on the assumption of simply supported rectangular plates and do not consider the boundary conditions existing in the flange–web junctions and in the stiffener–web junctions neither the geometry of the tapered steel plate girder. Some authors, among others Lee et al. [7], Mirambell and Zárate [8], Estrada et al. [9] have demonstrated the importance of these effects.

The ultimate shear strength models for tapered plate girders proposed in literature are based on previous presented models

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for plate girders with constant depth. Several models for tapered girders have been developed by: Falby and Lee [10], Davies and Mandal [11], Takeda and Mikami [12], Roberts and Newmark [13], Zárate and Mirambell [14] and Shanmugam and Min [15]. Recently, some other numerical studies have been published by Abu-Hamd and Abu-Hamd [16].

2.2. Ultimate shear strength for tapered plate girders

It is well known that the structural behaviour of a prismatic steel plate girder subjected to an increasing shear load up to failure may be divided into three clearly different phases. Prior to buckling, equal tensile and compressive principal stresses are developed in the web panel. In the post-buckling stage, an inclined tensile membrane stress state is developed. The total stress state is obtained by adding the post-buckling to that induced at buckling. Once the web has yielded, failure of the steel plate girder occurs when plastic hinges are formed in the flanges. The failure load can be determined from the consideration of the mechanism developed in the last stage (upper bound solution) or by the consideration of the equilibrium of forces (lower bound solution [5]).

The behaviour of a tapered steel plate girder subjected to increasing shear load is practically identical to that exhibited in a prismatic steel girder. When the web buckles under the action of direct stresses, it does not exhaust the full capacity of the plate. After buckling, a significant increase in the strength of the steel plate girder can be observed. Experimental tests and numerical studies carried out on tapered steel plate girders reveal the existence of post-critical strength, by means of the development of the diagonal tension field anchored in the stiffeners and flanges.

Some models for the determination of the ultimate shear strength for tapered plate girders have been presented in the last years. All these studies are based on the tension field method, but one determines the ultimate shear load by the lower (equilibrium) bound method [14], the other one by the upper (mechanism) bound method [15] and other one by both methods [11].

It is important to explain here the limitation of the existing methods. The proposal of Zárate and Mirambell [14] was thought only for these cases where the diagonal tension field develops in the short geometrical diagonal of the web panel. On the other hand, although the models proposed in [15] distinguish two different design situations, where inclined flange is in tension or in compression, there is no difference about the direction of the tension field.

In order to evaluate the methods abovementioned, a numerical study for rectangular and tapered plate girders was conducted. Research included various geometrical parameters and both situations, where the inclined flange is subjected to tension or compression, were analysed in [17]. The main conclusion of this study was that further ultimate shear models for tapered steel plate girders need to be developed in order to accurately evaluate the actual behaviour of tapered plate girders subjected to shear loads and their post-buckling resistance.

2.3. Shear resistance according to EN 1993-1-5

Despite being a very common type of beams, there are no specific rules for tapered plate girders in current codes. For calculating ultimate strength of tapered plate structures, EN 1993-1-5 suggests to use the expressions for prismatic plates without any changes if the angle of the inclined flange is not greater than 10° . In other cases is recommended to calculate a tapered plate as a rectangular plate with its larger depth. As it is shown in this paper, this approach cannot be used for some cases because overestimates the ultimate strength and thereby do not satisfy the safety requirements.

2.4. Four typologies – general behaviour

Previous numerical studies presented in [17,18] demonstrated that both critical load and ultimate strength of tapered plate girder are strongly influenced by two factors: (1) inclination of the flange and whether the flange is under tension or compression and (2) the direction of the developed tension field, which may appear on the short or on the long web diagonal. As a result it is possible to distinguish four different typologies of tapered plate girders (see Fig. 1):

- I. inclined flange in compression and diagonal tension field developed in the short diagonal;
- II. inclined flange in tension and diagonal tension field developed in the long diagonal;
- III. inclined flange in tension and diagonal tension field developed in the short diagonal;
- IV. inclined flange in compression and diagonal tension field developed in the long diagonal.

Fig. 2 shows examples of the application of each typology. Three of them can be met in various parts of a bridge span with non-prismatic cross-section, depending on the distribution of internal forces. The most common case is the first one, which appears frequently near to the intermediate supports of continuous bridges or in portal frames.

2.4.1. Resal effect

Different behaviour of each typology is provoked by appearance of an additional vertical component derived from the axial force in the inclined flange. This phenomenon is called Resal effect and can be favourable or not.

For these cases where the moment of inertia of the cross-section increases with the increase of internal forces (typology I and II), the vertical component acts against shear force and reduces it, thus the ultimate shear resistance is greater (positive influence). For typologies III and IV the opposite situation is observed. Graphical illustration of the Resal effect is presented in Fig. 3.

3. Experimental tests

Four small-scale experimental tests of tapered steel plate girders were carried out in the Laboratory of Structural Technology of the Polytechnic University of Catalonia to study the behaviour of tapered steel plate girders subjected to shear and shear-bending interaction. Experimental results were compared with those obtained by numerical simulation of the tests. All tested girders belonged to the same typology “I” which is considered as the most common case due to its geometry and the highest shear resistance (favourable Resal effect).

3.1. Geometry

The geometry of the tested girders with the symbols is presented in Fig. 4. All tested specimens had the same web thickness t_w and the same larger depth h_1 . They differ in the aspect ratio $\alpha = a/h_1$ ($\alpha = 1$ and 1.5) and the slope of the inclined flange $\tan(\phi)$ ($\tan(\phi) = 0.25$ and 0.4). Dimensions of the analysed specimens are presented in Table 1. Terminology for girder's name is following: e.g. A_600_800_800_4_180_15 means: $h_0 = 600$ mm, $h_1 = 800$ mm, $a = 800$ mm, web thickness $t_w = 4$ mm, flange width $b_f = 180$ mm and flange thickness $t_f = 15$ mm (see Fig. 4).

Girders A, B and C (see Fig. 4 and Table 1) were tested as simply supported, rigid end posted, short beams with the load applied at the mid-span to consequently obtain a constant shear law. Girder

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