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Numerical study of steel plate girders under shear loading at elevated temperatures

André Reis, Nuno Lopes *, Paulo Vila Real

RISCO — University of Aveiro, Department of Civil Engineering, Campus Universitário de Santiago, 3810-193 Aveiro, Portugal

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ABSTRACT

The ultimate shear capacity of steel plate girders may be influenced by shear buckling, an important and common instability phenomenon occurring in plate girders with slender webs. The transverse loads and consequent shear effort applied on such beams have a large impact on the webs, leading to the possible collapse by shear buckling. In a fire situation, the buckling phenomena in these structural elements are amplified due to the reduction of the steel mechanical properties caused by the elevated temperatures. Recently, there has been an increase in the use of those plate girders, arising from the search for more economical and competitive solutions. However, no specific rules for shear buckling verification in case of fire are given in Eurocode 3 (EC3).

After the validation of the numerical models against experimental tests from the literature, an extended number of geometrically and materially nonlinear numerical analyses including imperfections (GMNIA) have been performed dealing with the evaluation of the effect of rigid and non-rigid end posts on the ultimate shear capacity of steel plate girders under fire. The obtained numerical results are compared with the specifications of EC3 for the design at normal temperature adapted to elevated temperatures. These specifications are non-conservative and, for this reason, a new design proposal harmonized with the EC3 principles is presented in order to evaluate the ultimate shear capacity of steel plate girders in fire situation.

1. Introduction

Plate girders are commonly used in the construction industry, mainly in bridges and buildings with a high occupancy rate of people, as for example, sports arenas or shopping centres, due to their capacity to support high loads over long spans. These kinds of buildings have higher fire resistance requirements owing to the higher risk of loss of human lives. Plate girders usually have slender webs that are extremely susceptible to instability phenomena, particularly shear buckling.

Local buckling phenomena are very important in the design of steel structures composed of thin-walled cross-sections. For that reason, these have been a common topic in several research works. However, local buckling in structural elements subjected to fire has not received the same attention and in fact, fire is unfortunately a more common action in buildings than one would first think.

Shear buckling is a type of local buckling caused by the shear effort which was firstly studied by Wagner in 1931 [1] to evaluate the post-buckling shear strength of panels used in aircraft structures. In the late 1950s, investigations conducted by Basler and Thürlimann [2] led the American Institute of Steel Construction (AISC) to adopt the post-buckling strength into its specifications [3]. Later, investigations developed by Hoglund [4] led to the proposal of the Rotated Stress Field Method [5], which is the basis of the methodology adopted in Part 1–5 of Eurocode 3 (EC3) [6] to check the ultimate resistance of plate girders subjected to shear buckling.

Kang-Hai Tan and Qian Z.H. in 2007 [7] published a study on the shear buckling phenomenon at elevated temperatures. An experimental investigation of twelve simply supported steel plate girders subjected to shear loading at elevated temperature was carried out. It was observed that the ultimate shear capacity decreased significantly under a thermal restraint effect. The web panels of the plate girders were modelled using finite elements and a theoretical model to predict the failure loads of plate girders with transverse stiffeners subjected to a specified uniform temperature or isothermal condition as proposed by Vimonsatit V. et al. [8].

A numerical study about thin steel plates loaded in shear at non-uniform elevated temperatures was performed by Salminen and Heinisuo [9]. This study presented the results of a finite element analysis of 12 steel plates at 18 non-uniform temperatures and it led to the proposal of a new design method for predicting the shear resistance of thin steel plate at non-uniform elevated temperatures.

Recently, Scandella et al. [10] have shown that steel plate girders which fail at normal temperature due to flange buckling caused by the bending moment, may collapse at elevated temperatures due to web buckling caused by shear. In fact, large differences between the flanges and the web thicknesses can lead to a faster heating of the web, resulting in the occurrence of thermally induced compressive stresses

* Corresponding author.
E-mail address: nuno.lopes@ua.pt (N. Lopes).

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Table 1
Reduction factor from the web contribution to shear buckling.

	χ_w	
	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0.83/\eta$	η	η
$0.83/\eta \le \bar{\lambda}_w < 1.08$	$0.83/\bar{\lambda}_w$	$0.83/\bar{\lambda}_w$
$\bar{\lambda}_w \ge 1.08$	$1.37/(0.7+\bar{\lambda}_w)$	$0.83/\bar{\lambda}_w$

in the web that did not exist at normal temperature. In addition, Kodur and Naser [11] found that shear capacity can degrade faster than flexural capacity meaning that the shear limiting state may be a dominant failure mode in steel plate girders subjected to fire.

Although these studies provide valuable information on the behaviour of steel plate girders subjected to shear buckling at elevated temperatures, there is a lack of information regarding the influence of different parameters, such as the type of end posts, in the behaviour of steel plate girders with slender cross-sections in case of fire. Moreover, no specific rules for the ultimate shear strength verification at elevated temperatures are given in Part 1–2 of EC3 [12].

Therefore, the current work presents a numerical study on the behaviour at elevated temperatures of steel plate girders with strong flanges and slender webs highly affected by shear buckling. Parametric studies were carried out to study the influence of the rigid end post condition in the ultimate shear strength of plate girders, covering a wide range of slendernesses, aspect ratios and steel grades (S235, S275, S355 and S460).

The numerical modelling was performed using the finite element programme SAFIR (developed at University of Liège). The influence of the initial imperfections was taken into account, considering the geometric imperfections and the residual stresses. For the validation of the numerical model, comparisons between experimental results from the literature and the corresponding obtained numerical results were made. Afterwards, the numerical results from the parametric study are discussed and compared with the analytical methods implemented in EC3 adapted at elevated temperatures. Finally and due to the non-conservative nature of the results, new design expressions to determine the ultimate shear capacity in steel plate girders under fire are proposed.

2. Eurocode design methods

2.1. Resistance to shear

The behaviour of plates under shear involves two phenomena: the state of pure shear stress and the tension field action. Main tension field theories have been developed to determine the resistance of plates under shear [13,14]. In 1972, Torsten Hoglund developed a method called Rotated Stress Field Method [4] that lately was adopted in Part 1–5 of EC3 [6] with some modifications. Originally this method was developed for girders with web stiffeners at the supports only, because other existing methods were very conservative for this case [5]. In Part 1–5 of EC3 the Rotated Stress Field Theory was generally accepted, but the design expressions also include the flange contribution in the resistance mechanism. Furthermore, the behaviour of beams, with rigid and non-rigid end posts, has been taken into consideration for the evaluation of the contribution from the web to the shear buckling resistance. Those girders with rigid end posts are supposed to reach higher ultimate loads [15,16].

The method adopted in Part 1–5 of EC3 considers the shear resistance ($V_{b,Rd}$) as a sum of the resistance from the web to shear buckling ($V_{bw,Rd}$) and the flanges contribution ($V_{bf,Rd}$). However, the whole shear resistance ($V_{b,Rd}$) cannot be higher than the plastic shear resistance of the web alone, as shown in Eq. (1) [6].

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \le h_w\, t_w\, \frac{\eta\, f_{yw}}{\sqrt{3}\,\gamma_{M1}} \tag{1}$$

According to EN 1993-1-5, the resistance to shear buckling has to be verified when the ratio between the web depth and the web thickness (h_w/t_w) is higher than $72\,\frac{\varepsilon}{\eta}$ for unstiffened webs and $31\,\frac{\varepsilon}{\eta}\sqrt{k_\tau}$ for stiffened webs, ε, η and k_τ are given in EC3 [6]. In case these limits are exceeded, the girder should be provided with transverse stiffeners at the supports.

2.1.1. Contribution from the web

The contribution from the web is given by

$$V_{bw,Rd} = \chi_w\, h_w\, t_w\, \frac{f_{yw}}{\sqrt{3}\,\gamma_{M1}} \tag{2}$$

where χ_w is the reduction factor for the contribution of the web to shear buckling. The reduction factor is obtained as shown in Table 1 and is valid for both unstiffened and stiffened webs. The determination of this factor is in function of the slenderness of the web and depends on the end configuration (non-rigid or rigid end posts as shown in Fig. 1). The conditions for classifying a support as rigid or not are given in EN 1993-1-5 [6].

The recommended values for η according to EN 1993-1-5 [6] are $\eta = 1.2$ for steel grades up to $f_y = 460$ MPa and $\eta = 1.0$ for higher steel grades. Note that National Annexes of EC3 can give different values for η, also depending on the field of application.

The plate slenderness $\bar{\lambda}_w$ can be determined by Eq. (3). In case of a stiffened panel, the web slenderness value ($\bar{\lambda}_w$) to be used is the largest one of all sub-panels within the stiffened panel under consideration.

$$\bar{\lambda}_w = \sqrt{\frac{f_{yw}/\sqrt{3}}{\tau_{cr}}} = \frac{h_w}{37.4\, t_w\, \varepsilon\, \sqrt{k_\tau}} \tag{3}$$

The critical shear stress (τ_{cr}) may be determined according to formulas given in Part 1–5 of EC3 [6]. The calculation of the shear buckling coefficient can be done according to Annex A.3 of Part 1–5 of EC3. Otherwise, buckling charts and software tools may also be used. The critical stress is calculated considering hinged boundary conditions.

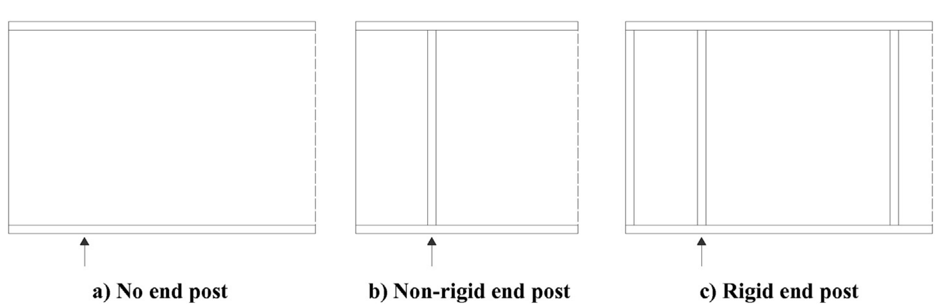

Fig. 1. End supports.