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Shear–bending interaction in steel plate girders subjected to elevated temperatures

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ABSTRACT

The purpose of the current study is to analyse the behaviour of steel plate girders with rigid and non-rigid end posts subjected to elevated temperatures, with the aim of assessing the interaction between shear and bending in case of fire. The intentional low stiffness of the flanges may precipitate the failure of these plate girders, making the interaction between shear and bending an important phenomenon which is also analysed in this work. A parametric numerical study was performed involving a wide range of cross-section's dimensions, plate girders' aspect ratios and steel grades. Plate girders were numerically tested at both normal and elevated temperature, considering three different uniform temperatures 350 °C, 500 °C and 600 °C. The influence of the geometrical imperfections, as well as the residual stresses, was taken into account. Finally, the numerical results were compared to the EC3 prescriptions for shear buckling and shear–bending interaction safety verifications, adapted to fire situation by the direct application of the reduction factors for the stress-strain relationship of steel at elevated temperatures.

1. Introduction

Recently there has been an increase in the use of steel plate girders, due to their capacity to support high loads over long spans. Thanks to their light weight, plate girders are economically competitive solutions which are commonly applied in buildings and bridges. For material efficiency, they usually have slender webs when compared to the ones on commercial hot rolled profiles, making them economically susceptible to the occurrence of instability phenomena, as for example shear buckling.

Shear buckling is a very important phenomenon on the design of steel structural elements with thin-walled cross-sections. Hence, it has been widely studied in the last decades leading to the implementation in Part 1–5 of Eurocode 3 (EC3) [1] of the Rotated Stress Field Method [2] for the normal temperature design of steel plate girders under shear. There are several parameters that influence the ultimate shear strength of steel plate girders subjected to shear buckling, such as the web slenderness, the use of transverse and longitudinal stiffeners and the type of end posts. Moreover, the interaction between shear and bending moment may also cause a significant reduction on the ultimate resistance of these types of girders [3,4]. The interaction between shear and bending in steel plate girders is a current topic (always at normal temperature) on the research activities of several authors. For instance, Graciano et al. [5] analysed the behaviour of steel plate girders without longitudinal stiffeners while Kovesdi et al. [6,7] extended the research to longitudinal stiffened girders too.

Regarding steel structural elements subjected to elevated temperature, shear buckling has not received the same attention. However, over the last years, there has been a greater concern with the shear buckling occurrence at elevated temperatures. Kodur and Naser [8] found that the failure by shear is the dominant failure mode in steel plate girders in fire situation. Moreover, it was found that the buckling phenomena on these types of structural elements are amplified under fire conditions, due to either the reduction of the steel mechanical properties caused by the elevated temperatures [9,10] or the appearance of additional compressive stresses resulting from the increase of temperature. A numerical study about thin steel plates loaded in shear at non-uniform elevated temperatures was performed by Scandella et al. [11] in which it was shown that the non-uniform temperatures can impose additional loading and even chance the failure mode. The large differences between the flanges and web thicknesses can lead to a faster heating in the web than flanges, resulting in the development of thermally induced compressive stresses in the web, which will accelerate the local failure. Thus, a steel plate girder with a bending dominant failure at normal temperature may instead exhibit a shear dominant failure at elevated temperatures with non-uniform heating.

A new design method for predicting the shear resistance of thin steel plate at non-uniform elevated temperatures has been proposed by Salminen and Heinisuo [12]. The basic idea of the method is to reduce the ultimate shear strength of the plate based

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Nomenclature

a	transverse stiffeners spacing
b_f	flange width
c	plastic hinges spacing
E	Young's modulus
f_{yf}	flange yield strength
f_{yw}	web yield strength
h_w	web depth
$k_{E,\theta}$	reduction factor for Young's modulus at elevated temperature
$k_{y,\theta}$	reduction factor of the steel yield strength at elevated temperature
$k_{0.2p,\theta}$	reduction factor of the flanges' resistance to the bending moment at elevated temperature used for Class 4 cross-sections
k_τ	shear buckling coefficient
M_{ed}	design bending moment
$M_{f,Rd}$	resistance moment of the cross-section consisting of the effective area of the flanges only
M_{SAFIR}	bending moment numerically obtained
t_w	web thickness
t_f	flange thickness
$V_{b,rd}$	shear resistance
$V_{b,fi,rd}$	shear resistance at elevated temperature
$V_{bf,rd}$	flanges contribution to shear buckling
$V_{bf,fi,rd}$	flanges contribution to shear buckling at elevated temperature
$V_{bw,rd}$	resistance from the web to shear buckling
$V_{bw,fi,rd}$	resistance from the web to shear buckling at elevated temperature
V_{SAFIR}	shear resistance numerically obtained
γ_{M1}	partial safety factor which is equal to 1.0
χ_f	factor for the flange contribution to shear buckling
χ_w	reduction factor for the contribution of the web to shear buckling
$\chi_{w,SAFIR}$	reduction factor for the contribution of the web to shear buckling numerically obtained
$\chi_{w,fi}$	reduction factor for the contribution of the web to shear buckling at elevated temperature
ε	coefficient depending on the steel grade
ε_θ	coefficient depending on the steel grade at elevated temperature
$\bar{\lambda}_w$	web slenderness parameter
$\bar{\lambda}_{w,\theta}$	web slenderness parameter at elevated temperature
η	coefficient to take into account the strain hardening
τ_{cr}	critical shear stress

on a reference temperature, which is hotter than the average temperature but colder than the maximum temperature. The authors suggested that non-uniform temperature distributions should be converted into an equivalent uniform temperature, which highlights the importance of using simple design methods giving safe predictions.

Despite this growing awareness of the possibility of occurrence of shear buckling during a fire, it has not yet been shown if the prescriptions adopted in Part 1–5 of EC3, for the plate girders design at normal temperature, may be directly applied to fire design. In this study, due to the lack of rules in Part 1–2 of EC3 [13] for the shear buckling verification at elevated temperatures, a simple methodology was used, based on the design rules at normal temperature, adapted to fire situation by the direct application of the reduction factors for the steel stress-strain relationship at elevated temperatures.

The main goal of this research is to study the shear response of steel plate girders with rigid and non-rigid end posts in fire situation, assessing whether the procedure from Part 1–5 of EC3 for the design at normal temperature may be adapted for fire design. For that purpose, a parametric numerical study was performed using a numerical model which was validated with experimental tests [14] in the FEM software SAFIR [15,16]. The influence of the geometric imperfections was taken into account considering the first buckling modes, which were obtained using CAST3M software [17]. For the interface between SAFIR and CAST3M, the computer programme named RUBY [18] was used. Furthermore, the residual stresses were also taken into account considering the typical pattern for welded I-sections.

Simply supported plate girders with nine different cross-sections were analysed. The shear buckling in steel plate girders with stronger flanges was previously evaluated [19]. In this study, flanges with low stiffness were chosen to allow the shear–bending interaction. Four different steel grades, as well as four different distances between transverse stiffeners were considered, which allowed studying a wide range of aspect ratios (a/h_w). A total of 144 plate girders with rigid end posts and 144 plate girders with non-rigid end posts were analysed at normal temperature and subjected to different uniform temperatures (350 °C, 500 °C and 600 °C) under steady-state conditions, i.e., the temperature is considered constant while the load is increased until failure. Finally, the numerical results are compared to the analytical ones obtained according to the procedures implemented in the EC3 for the safety verification of the ultimate shear strength of steel plate girders.

2. EC3 design method

Part 1–1 of EC3 [20] states that the ultimate shear strength of plate structural elements must be checked according to Part 1–5 of EC3 [1] when some requirements are fulfilled. In Part 1–5 of EC3, provisions for the safety verification of both shear buckling and interaction between shear and bending may be found.

2.1. Shear buckling

According to Part 1–1 of EC3 [20], the shear buckling resistance must be checked when the ratio between the web height and the web thickness satisfies one of the following conditions:

i. $h_w/t_w > 72\varepsilon/\eta$ for unstiffened webs
ii. $h_w/t_w > 31\varepsilon/\eta\sqrt{k_\tau}$ for stiffened webs

where ε is a parameter that depends on steel yield strength and on Young's modulus, η is a coefficient related to the strain hardening and k_τ is the shear buckling coefficient of the web plate. If these conditions are fulfilled it is still necessary to provide transverse stiffeners at the supports.

The Rotated Stress Field Method developed by Höglund is the basis of the shear design rules given in Part 1–5 of EC3 [1]. In the EN 1993-1-5 procedure, the ultimate shear resistance $V_{b,Rd}$ is given by Eq. (1) and expressed as the sum of the web shear buckling resistance $V_{bw,Rd}$ presented in Eq. (2) and the flange contribution $V_{bf,Rd}$ given by Eq. (3).