Shear design recommendations for stainless steel plate girders

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Abstract

The behaviour and design of stainless steel plate girders loaded in shear investigated in this paper. A review of existing methods for the design of stainless steel plate girders, including codified provisions, is first presented. A collected database of thirty four experiments carried out on stainless steel plate girders of the austenitic, duplex and lean duplex grades is then reported, and used to assess the current shear resistance design equations found in Eurocode 3: Part 1.4, Eurocode 3: Part 1.5 and those proposed by Estrada et al. The comparisons clearly indicate that the design provisions of Eurocode 3: Part 1.4 are conservative and that improved results can be achieved by applying the Eurocode 3: Part 1.5 and Estrada et al. design expressions. However, yet further improvements are possible and, based on the available structural performance data, revised design expressions for the calculation of the ultimate shear capacity of stainless steel plate girders suitable for incorporation into future revisions of Eurocode 3: Part 1.4 have been proposed and statistically verified. Unlike the current provisions of Eurocode 3: Part 1.4, the design rules proposed herein differentiate between rigid and non-rigid end posts, and, offer enhancements in shear buckling capacity of around 10%.

Keywords

Design methods, Eurocode 3, Experimental data, Failure modes, Plate girders, Reliability analysis, Rigid and non-rigid end post, Rotated stress field method, Shear buckling, Stainless steel, Ultimate shear capacity,.

1 Introduction

Plate girders are widely used in the construction industry especially in bridge applications, as transfer beams and shear walls in buildings and in offshore structures, owing to their ability to withstand heavy loads over long spans. For material efficiency, plate girder webs are often of slender proportions, making them susceptible to a form of instability known as shear buckling. This type of failure has been extensively studied over the past few decades in carbon steel plate girders and a range of design methods have been established. A more limited number of studies has been devoted to stainless steel plate girders, and current design provisions are known to be conservative.

Hence, the aims of this paper are to study the shear response of stainless steel plate girders, to collate and examine available structural performance data, to review existing design methods and to develop and statistically verify revised design expressions suitable for inclusion in international design codes. A total of thirty four experiments carried out on stainless steel plate girders of the austenitic, duplex and lean duplex grades, with web panel aspect ratios varying between 1.0 and 4.0 and with rigid and non-rigid end posts, were first collected and used to evaluate the shear resistance design equations of EN 1993-1-4 (2006) [1]. Next, a comparative analysis of other design methods including EN 1993-1-5 (2006) [2] and the proposed design expressions of Estrada et al. [3] has been performed. Finally, based on the available experimental data, together with supporting numerical data, revised design expressions for the calculation of the ultimate shear capacity of stainless steel plate girders are proposed and a reliability analysis in accordance with EN 1990 (2002) [4] was carried out to confirm their applicability.

2 Literature review

2.1 Introduction

In this section, laboratory test data on stainless steel plate girders and existing design methods and proposals for assessing the shear buckling resistance of plate girders are presented and briefly reviewed. The design methods discussed are the tension field method and the rotated stress field method.

2.2 Overview of previous experimental studies

The first experimental investigation of stainless steel plate girders was carried out by Carvalho et al. [5]. The results of this study served as the basis for the first codified proposals for determining the ultimate shear resistance of stainless steel beams in ENV 1993-1-4 (1996) [6]. Following the introduction of ENV 1993-1-4 (1996) [6], a series of experimental studies were carried out by Olsson [7], Real et al. [8] and Estrada et al. [9], which highlighted deficiencies in the existing design methods.. The main objectives of these investigations were to develop a better understanding of the behaviour of stainless steel plate girders under shear and to propose design expressions capable of predicting accurately the shear resistance of stainless steel plate girders. Most recently, a detailed experimental and numerical study of lean duplex stainless steel plate girders was conducted [10], bringing the total pool of laboratory test data to 34. The findings of these studies are discussed in Section 4 of this paper, while the experimental data are used to verify a revised design treatment.

2.3 Overview of theoretical models for predicting shear buckling resistance

The first attempt to estimate the shear resistance of slender plate girders was made by Basler et al. [11, 12]. According to Basler, once shear buckling had occurred in a plate girder web, a theoretical tension field would extend over the whole depth of the web and the shear resistance could be expressed as the sum of the buckling and postbuckling resistances of the web but with no flange contribution. Although there were limitations to the Basler theory [13, 14], the basic tension field concept was able to represent observed physical behaviour and was further developed by Rockey et al. [15, 16]. A drawback to the tension field approach lies in its inability to predict accurately the shear buckling resistance of plate girders with widely spread transverse web stiffeners. A solution, termed the rotated stress field method [7], was proposed by Höglund [17, 18]. This method was able to represent the postbuckling shear strength of both stiffened and unstiffened webs.

The rotated stress field method assumes that, prior to buckling, the web is in a state of pure shear stress and the principal planes are inclined at an angle $\varphi = 45°$ to the horizontal. However, once buckling has occurred, it is assumed that the principal compressive stress, remains equal to the shear buckling stress τ_{cr}, and that further increases in load are resisted by an increase in the tensile stress. This causes the major principal plane to rotate towards the horizontal, and the ultimate resistance is said to be reached when the von Mises yield criterion [7] is met. A detailed description of the rotated stress field theory can be found in [19].

3 Current and proposed design methods

3.1 Introduction

In this section, the design methods for assessing the shear buckling resistance of plate girders in Eurocode 3 are reviewed, including the evolution from the ENV prestandard to final EN standard. The provisions for both carbon steel and stainless steel, as well as proposed changes to the latter, are considered.

3.2 Carbon steel design provisions

3.2.1 ENV 1993-1-1 (1992)

Two methods were provided in ENV 1993-1-1 (1992) [20] for determining the design shear resistance of carbon steel plate girders: (1) the simple post critical method and (2) the tension field method. The first design method was developed by Dubas [21] based on the rotated stress field theory, and applied to plate girders with and without transverse stiffeners. The design shear resistance ignored any flange contribution, and was later found by Höglund [19] and Davies and Griffith [22], to be unduly conservative. The second method, the tension field method, was found to be only appropriate for transversely stiffened webs with web panel aspect ratios ranging between 1.0 and 3.0 [23]. Furthermore, numerical studies by Presta et al. [24] showed that the forces in the transverse stiffeners implied by the tension field method were inaccurate. Limitations in both methods given in ENV 1993-1-1 (1992) [20], lead to revised design rules being provided upon conversion to the EN standards, at which point the design provisions for shear buckling were also moved to Part 1.5 of the code.

3.2.2 EN 1993-1-5 (2006)

The rotated stress field method developed by Höglund [17, 18] forms the basis of the shear design rules given in EN 1993-1-5 (2006) [2]. In the EN 1993-1-5 (2006) [2] provisions, ultimate shear resistance $V_{b,Rd}$ is expressed as the sum of the web shear buckling resistance $V_{bw,Rd}$ (Eq. (2)) and the flange contribution (Eq. (3)) $V_{bf,Rd}$, as set out in Eq. (1).

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

where f_{yw} is the yield strength of the web, f_{yf} is the yield strength of the flanges, η is a parameter that approximates the influence of strain hardening, h_w is the depth of the web, t_w is the thickness of the web, b_f is overall the flange width, t_f is the flange thickness, and γ_{M1} is a partial safety factor.

The web contribution $V_{bw,Rd}$ is given by

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

where χ_w is the web shear buckling reduction. The flange contribution $V_{bf,Rd}$ is given by:

$$V_{bf,Rd} = \left(\frac{b_f t_f^{\,2} f_{yf}}{c\gamma_{M1}}\right)\left(1-\left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right) \tag{3}$$

in which M_{Ed} is the coexistent design bending moment, $M_{f,Rd}$ is the moment resistance of the cross-section considering only the flanges and the distance c which defines the location of the plastic hinges that form in the flanges is given by:

$$c = \left(0.25 + \frac{1.6 b_f t_f^{\,2} f_{yf}}{t_w h_w^{\,2} f_{yw}}\right)a \tag{4}$$

where a is the spacing of the transverse stiffeners.

3.3 Stainless steel design provisions

3.3.1 ENV 1993-1-4 (1996)

At the time of the development of ENV 1993-1-4 (1996) [6], the only experimental research into the shear resistance of stainless steel members was that carried out by Carvalho et al. [5]. Carvalho et al. [5] performed a series of three point bending tests on cold-formed austenitic and ferritic stainless steel sections. The obtained results were used in the formulation of the design provisions of ENV 1993-1-4 (1996) [6], which were based on the simple post critical method of ENV 1993-1-1 (1992) [20], but with modifications to reflect the material nonlinearity of stainless steel. Subsequent experimental studies on stainless steel plate girders [3, 7-9] showed that the ENV 1993-1-4 (1996) [6] provisions were conservative, raised questions of the quality of the earlier test data [5] and emphasized the need to consider the flange contribution to the shear buckling capacity.

3.4 EN 1993-1-4 (2006)

Following the adoption of the simple post critical method in ENV 1993-1-4 (1996) [6], Olsson [7] performed an experimental and analytical study to underpin the development of improved new design expressions for stainless steel plate girders. Olsson's design equations were based on the rotated stress field method, took into consideration both the web and flange contributions to the shear resistance, and were of the same basic form as EN 1993-1-5 (2006) [2], given by Eqs (1-3) of the present paper. Deviation from the EN 1993-1-5 (2006) [2] provisions appeared in the expressions for the shear buckling reduction factor χ_w and in the definition of the distance c. In Olsson's [7] proposal $\chi_w = \eta$ for $\bar{\lambda}_w \leq 0.6/\eta$ and $\chi_w = 0.11 + 0.64/\bar{\lambda}_w - 0.05/\bar{\lambda}_w^2$ for $\bar{\lambda}_w > 0.6/\eta$, with $\eta = 1.2$, and c is defined by Eq. (5).

$$c = \left(0.17 + \frac{3.5 b_f t_f^{\,2} f_{yf}}{t_w h_w^{\,2} f_{yw}}\right)a \quad \text{with} \quad \frac{c}{a} \leq 0.65 \tag{5}$$

Olsson's method was included in the second edition of the SCI/Euro Inox Design Manual for Structural Stainless Steel [25] and was later incorporated into the final version of EN 1993-1-4 (2006) [1].

3.5 Estrada et al.'s proposal (2007)

Olsson's design expressions offered clear benefits over those given in ENV 1993-1-4 (1996) [6]. However, the improved rules still did not distinguish between rigid and non-rigid end posts. Hence, further research was carried out by Estrada et al. [3, 9] where the influence of end post rigidity was evaluated over a wide range of web panel aspect ratios and slendernesses. Based on their findings, revised design expressions were proposed.

The proposed expressions were again based on the rotated stress field method and the total shear buckling resistance comprised a web and flange contribution, as set out in Eqs (1), (2) and (3). The flange contribution was the same as that proposed by Olsson, while the web contribution was revised. As explained by Estrada et al. [3], different design expressions were given for web panel aspect ratios less than and greater than unity. For the former case, end post rigidity was taken into account, providing more accurate prediction of test behaviour.

4 Analysis of structural performance data on stainless steel plate girders

4.1 Introduction

In order to evaluate the provisions outlined in Section 3 for the design of stainless steel plate girders, the results from previously conducted laboratory tests on stainless steel plate girders were collected and analysed. In this section, the available test data are compared to the shear design rules of EN 1993-1-4 (2006) [1], EN 1993-1-5 (2006) [2] and those proposed by Estrada et al. [3].

4.2 Collected experimental data on stainless steel plate girders

A total of 34 experiments on stainless steel plate girders have been conducted- see Table 1. Of these, 21 (labelled NR1 to NR21) had non-rigid end posts and 13 (labelled R1 to R13) had rigid end posts. The tested web panel aspect ratio a/h_w ranged between 1.00 and 3.25, while the non-dimensional web slenderness ranged between 0.44 and 3.34. The collected experimental data is summarised in Table 1, where L is the specimen length, a is the shear panel length, b is the overall flange width, h_w is the web depth, t_f is the thickness of the flange, t_w is the thickness of the web, t_{s1} and t_{s2} are the thicknesses of the stiffeners, $\sigma_{0.2,w}$ and $\sigma_{0.2,f}$ are the 0.2% proof stresses of the web and flanges, a/h_w is the aspect ratio of the web panel, $\bar{\lambda}_w$ is the non-dimensional web slenderness and $V_{u,test}$ is the ultimate shear capacity from the experiments. The definitions of those symbols are also illustrated in Fig. 1.

4.3 Analyses of results

4.3.1 General

In analysing the test results, two cases were considered - case 1: plate girders exhibiting a shear dominant failure defined as those where the ratio of shear force to bending moment in the test $V_{u,test}/M_{u,test} > V_{bw,Rd}/M_{f,Rd}$ and case 2: plate girders exhibiting a bending dominant failure or a combined bending plus shear failure (i.e. $V_{u,test}/M_{u,test} \leq V_{bw,Rd}/M_{f,Rd}$). These two cases are illustrated in Fig. 2. Test results in case 1 only were used to assess the shear buckling resistance design expressions, while all data (cases 1 and 2) were used to investigate the provisions for moment-shear interaction. The collected test data are plotted in Fig. 3, together with the normalised moment-shear interaction diagram, calculated according to EN 1993-1-4 (2006) [1]. Three interaction curves are shown in Fig. 3. All have the same form, but since the precise shape of the curves varies with $V_{bw,Rd}/V_{bf,Rd}$ and $M_{f,Rd}/M_{c,Rd}$, and since these ratios are different for each test specimen, clearly there is no single curve against which to compare. The three curves shown are for the average, minimum and maximum values of the two ratios. Note that, in all comparisons shown, the measured geometric and material properties from the test specimens are used, and all partial safety factors are set equal to unity. Furthermore, in the following sections of this paper, case 1 and 2 plate girders were identified by studying the moment-shear interaction diagram for each cross-section separately.

4.3.2 Comparison of existing test data with EN 1993-1-4 (2006)

The results collected from the experiments carried out on stainless steel plate girders were used to evaluate the current shear design provisions of EN 1993-1-4 (2006) [1]. The test results of the plate girders exhibiting a shear dominant failure (i.e. case 1 only) are plotted with the EN 1993-1-4 design model in Fig. 4, in terms of shear buckling reduction factor χ_w versus web slenderness $\bar{\lambda}_w$. A distinction has been made between the tested plate girders with rigid and non-rigid end posts, though EN 1993-1-4 (2006) [1] makes no such distinction. In locating the test data points, the flange contribution calculated according to Eq. (5), has been deducted from the test ultimate shear resistance, and the result has been normalised by the yield capacity of the web in shear. The high normalised shear capacities (beyond yield and indeed beyond 1.2 times yield) obtained at low slendernesses are attributed to strain hardening.

For assessing the design provisions for moment-shear interaction, all test data (i.e. cases 1 and 2) were considered. Assuming proportional loading (i.e. the ratio of shear force to bending moment remains constant), the ratio by which each test data point exceeded or fell short of its respective design interaction curve was denoted U. A value of U greater than unity indicates a safe result whereby the test data point lies outside the interaction curve. The results are shown in Table 2 and illustrated in Fig. 5. The comparisons show that the test results consistently lie above the codified predictions, with a mean ratio of $U_{EN\ 1993\text{-}1\text{-}4}$ of 1.22 and a coefficient of variation (COV) of 0.11.

4.3.3 Comparison of existing test data with EN 1993-1-5 (2006)

The European design standard for carbon steel plate girders EN 1993-1-5 (2006) provides shear design expressions of the same form as EN 1993-1-4 (2006), but with alternative coefficients to reflect the differences in material response.

Unlike EN 1993-1-4, EN 1993-1-5 differentiates between rigid and non-rigid end post plate girders for web slenderness values $\bar{\lambda}_w > 1.08$.

As in Section 4.3.2, the test results that fall into case 1 are used to compare with the codified shear buckling resistance predictions (see Fig. 6), while results for both cases 1 and 2 are used to assess the moment-shear interaction (see Table 2). The comparisons show that the predictions of EN 1993-1-5 (2006) [2] are noticeably higher than those of EN 1993-1-4 (2006) [1] and provide better agreement with the test results, with mean utilisation ratio $U_{EN\ 1993\text{-}1\text{-}5}$ of 1.13 and a coefficient of variation (COV) of 0.14.

4.3.4 Comparison with Estrada et al. [3] proposed design equations

Similar comparisons to those described in Sections 4.3.2 and 4.3.3 are made in the present section with the proposals of Estrada et al. [3]. Unlike EN 1993-1-4 (2006), Estrada et al.'s [3] design expressions differentiate between rigid and non-rigid end post plate girders when the web panel aspect ratio is less than unity, while for higher aspect ratios, revised design expressions are also provided.

Test results for case 1 only are compared with the Estrada et al. [3] proposals for shear buckling resistance in Fig. 7, while the utilisation ratio for all test results, considering combined bending and shear are shown in Table 2. The comparisons show that Estrada et al.'s predictions are higher than those of EN 1993-1-4 (2006) [1] and provide better agreement with the test results, with a mean utilisation ratio $U_{Estrada}$ of 1.12 and a coefficient of variation (COV) of 0.14.

4.4 Discussion

Collected test data on stainless steel plate girders have been compared to the design provisions of EN 1993-1-4 (2006) [1], EN 1993-1-5 (2006) [2] and the proposed equations of Estrada et al. [3]. In general, it is observed that the results obtained from the estimations of EN 1993-1-4 (2006) [1] are conservative and better results can be obtained from EN 1993-1-5 (2006) [2] and the Estrada et al. [3] design expressions. Further improvements to the provisions, considering the recently available test and numerical data [10], are proposed in the following section.

5 Design proposals

5.1 Introduction

The comparisons of the previous section show that the current provisions of EN 1993-1-4 (2006) [1] are conservative and better predictions of the test response are achieved with the proposed design equations of Estrada et al. [3] or those of EN 1993-1-5 (2006) [2]. In this section, further improvements are sought and new design expressions for the calculation of the ultimate shear capacity of stainless steel plate girders are proposed. Statistical analyses, in accordance with EN 1990 [4], are also carried out to assess the reliability of the proposals.

5.2 Proposed design method

Based on the collected test data reported herein and the numerically generated plate girders from Saliba and Gardner [10], new design expressions are proposed to predict the ultimate shear capacity of stainless steel plate girders. The proposed expressions follow the same approach of EN 1993-1-4 (2006) [1] and EN 1993-1-5 (2006) [2] in which the ultimate shear capacity of a stainless steel plate girder, $V_{b,Rd}$, consists of a web contribution $V_{bw,Rd}$ and a flange contribution $V_{bf,Rd}$, as given by Eq. (1).

The flange contribution $V_{bf,Rd}$ in the proposed approach is taken to be the same as that currently given in EN 1993-1-4 (2006) [1] – see Eq. (3). The basis for this is that the expression (Eq. (5)) for the controlling parameter, c, which defines the location of the plastic hinges in the ultimate shear collapse mechanism, has been confirmed to provide an accurate representation of recently generated test data [3, 9, 10], as well as Olsson's test data [7] upon which the expression was originally derived. The web contribution to shear resistance $V_{bw,Rd}$, given by Eq. (2), is controlled by the shear buckling reduction factor χ_w. Here, the proposed design expressions differ from previous provisions. The proposed expressions are developed on the following basis: (1) a larger body of test data (totalling 34 experiments) is considered than has previously been available, (2) a distinction is made between rigid and non-rigid end posts over the full spectrum of web panel aspect ratios and (3) a reliability analysis in accordance with EN 1990 [4] is conducted.

The proposed shear buckling reduction factors χ_w are presented in Table 3. The maximum value of χ_w adopted in the proposed design equations is the same as that of EN 1993-1-4 (2006) [1], at a value of 1.2. This approximates the extra capacity beyond yield as a result of strain hardening.

The results collected from the experiments on stainless steel plate girders were used to assess the proposed equations. The utilisation ratios $U_{Proposed}$ (considering cases 1 and 2) are shown in Table 2 and Fig. 8 with a mean value of 1.12 and a coefficient of variation (COV) of 0.13. Note that the EN 1993-1-5 (2006) [2] moment-shear interaction expressions is retained. In general, the utilization ratios for the proposed equations are significantly lower than those of EN 1993-1-4 (2006) [1], indicating better estimations of the ultimate shear resistance of the tested plate girders. The results of the case 1 plate girders are also plotted with the proposed design model in Fig. 9 where a better agreement with the test data can be observed. Further statistical analysis is performed in the following section to verify the reliability of the proposed design expressions.

5.3 Reliability analysis

The aim of the statistical analysis carried out in this section is to verify whether the developed design equations for the calculation of the shear resistance of stainless steel plate girders satisfy the Eurocode reliability requirements. The analysis followed the standard statistical evaluation set out in Annex D of EN 1990 (2002) [4] and was applied to the collected test data reported herein and the numerically generated results from Saliba and Gardner [10]. The analysis focused on shear buckling resistance and therefore considered the case 1 results only.

A key assumption of the standard evaluation procedure is that the resistance expression is a function of independent variables. The dominant component of the shear buckling resistance (i.e. the web shear buckling function resistance) $V_{bw,Rd}$, which generally appears in the form of Eq. (2) may alternatively be expressed as

$$V_{bw,Rd} = X\tau_{yw}^{a}A_{w}^{b} \tag{6}$$

where $f_{\tau w}$ is equal to $\tau_{yw}/\sqrt{3}$ and $A_w = h_w\, t_w$ are the two independent variables and X is a constant, which does not depend on the other two parameters. The powers a and b vary for different slenderness and should be determined for each test specimen. The power a is calculated by assuming two plate girders of the same geometrical properties but with different material strengths $\tau_{yw,1}$ and $\tau_{yw,2}$. The ratio of the resistances of the two considered plate girders is given by Eq. (7):

$$\frac{V_{bw,Rd,2}}{V_{bw,Rd,1}} = \frac{X\tau_{yw,2}^{a}A_{w}^{b}}{X\tau_{yw,1}^{a}A_{w}^{b}} = \left(\frac{\tau_{yw,2}}{\tau_{yw,1}}\right)^{a} \tag{7}$$

Therefore, the power a may be calculated as follows:

$$a = \frac{\ln\left(\frac{V_{bw,Rd,2}}{V_{bw,Rd,1}}\right)}{\ln\left(\frac{\tau_{yw,2}}{\tau_{yw,1}}\right)} \tag{8}$$

and the power b may be subsequently obtained from:

$$b = \frac{\ln\left(\frac{V_{bw,Rd,2}}{V_{bw,Rd,1}}\right) - a\ln\left(\frac{\tau_{yw,2}}{\tau_{yw,1}}\right)}{\ln\left(\frac{A_{w,2}}{A_{w,1}}\right)} \tag{9}$$

by considering two plate girders of different web areas $A_{w,1}$ and $A_{w,2}$.

The relationship between the two powers a and b and the non-dimensional web slenderness $\bar{\lambda}_w$ is plotted in Fig. 10. The values of the parameters are calculated based on the proposed equations in Section 5.2. At low slenderness ($\bar{\lambda}_w \leq 0.56$), χ_w equals 1.2 and $V_{bw,Rd} = 1.2\tau_{yw}A_w/\gamma_{M1}$, with a = b = 1, while at high slendernesses, where the resistance is governed by buckling and post-buckling tension field action, the values of a and b alter in response to the varying influence of τ_{yw} and A_w.

In order to properly allow for the influence of the basic variables τ_{yw} and A_w at different values of web slenderness in the reliability analysis, the variability of τ_{yw} and A_w needs to be modified. The coefficient of variation of the basic variables, V_{rt}, for a resistance function that is not simply linearly dependent on the basic variables may be obtained according to Eq. (D.16b) from Annex D of EN 1990 (2002) [4] and presented as follows:

$$
\begin{aligned}
V_{rt}^2 &= \frac{VAR[g_{rt}(X)]}{g_{rt}^2(X_m)} \cong \frac{1}{g_{rt}^2(X_m)} \sum_{i=1}^{j} \left(\frac{\partial g_{rt}}{\partial X_i} \sigma_i \right)^2 \\
&= \frac{1}{g_{rt}^2(X_m)} \left[\left(\frac{\partial g_{rt}}{\partial \tau_{yw}} \sigma_{f_{\tau w}} \right)^2 + \left(\frac{\partial g_{rt}}{\partial A} \sigma_{A_w} \right)^2 \right] \\
&= \frac{1}{g_{rt}^2(X_m)} \left[\left(a \tau_{yw}^{a-1} A_w^b \sigma_{\tau_{yw}} \right)^2 + \left(b \tau_{yw}^a A_w^{b-1} \sigma_{A_w} \right)^2 \right] \\
&= \frac{1}{g_{rt}^2(X_m)} \left[\left(\frac{a g_{rt}}{\tau_{yw}} \sigma_{\tau_{yw}} \right)^2 + \left(\frac{b g_{rt}}{A_w} \sigma_{A_w} \right)^2 \right] \\
&= \frac{g_{rt}^2(X_m)}{g_{rt}^2(X_m)} \left[\left(a \frac{\sigma_{\tau_{yw}}}{\tau_{yw,m}} \right)^2 + \left(b \frac{\sigma_{A_w}}{A_{w,m}} \right)^2 \right] = \left(a V_{\tau_{yw}} \right)^2 + \left(b V_{A_w} \right)^2
\end{aligned}
\tag{10}
$$

where $VAR[g_{rt}(X)]$ and $g_{rt}(X_m)$ are the variance and the mean, respectively, of the resistance function $g_{rt}(X)$, $\sigma_{\tau yw}$ and σ_{Aw} are the standard deviations of the yield strength and web area, respectively, $\tau_{yw,m}$ and $A_{w,m}$ are the mean values of the shear yield strength and web area, respectively and $V_{\tau yw}$ and V_{Aw} are the coefficients of variation of the shear yield strength and web area, respectively. This modification was incorporated into Step 7 of the statistical analysis set out in Annex D of EN 1990 (2002) [4].

A statistical evaluation based on the above described modified approach was then performed for the collected test and numerical data. Certain statistical parameters were assumed based on previous studies of the mechanical and geometrical properties of structural sections. The ratio of mean to nominal yield strengths (i.e. the material over-strength) was taken as 1.33 for austenitic stainless steels and as 1.2 for duplex stainless steels and the coefficients of variation of yield strength $V_{\tau yw}$ and geometric properties V_{Aw} were taken as 0.066 and 0.025 for austenitic stainless steel and 0.04 and 0.025 for duplex stainless steel, respectively [26, 27]. These values originate from industrial data obtained from European steel producers.

The results of the analyses and a summary of the key statistical parameters are presented in Table 4. The following symbols are used: $k_{d,n}$ = design (ultimate limit state) fractile factor for n tests, where n is the population of test data under consideration, b = average ratio of experimental to model resistance based on a least squares fit to the test data; V_δ = coefficient of variation of the tests relative to the resistance model; and V_r = combined coefficient of variation incorporating both model and basic variable uncertainties. Note that in accordance with EN 1990 (2002) [4], the fractile factor for the full collection of data on stainless steel plate girders has been used in the statistical analysis. Considering the resistance function for austenitic stainless steel plate girders, the partial factor γ_{M1} was found to be 1.04 for the test data only and for the resistance function of lean duplex stainless steel plate girders, the partial factor γ_{M1} was found to be 1.06 for the test data plus FE results of Saliba and Gardner [10]. The obtained values of γ_{M1} for both austenitic and duplex/lean duplex stainless steel are less than 1.1, which is the recommended partial factor for stainless steel shear buckling in EN 1993-1-4 (2006) [1]. It is therefore recommended that the proposed equations for the calculation of the web shear resistance in Section 5.2 can be safely applied to stainless steel plate girders, with γ_{M1} = 1.1.

6 Conclusions

The behaviour of stainless steel plate girders, with an emphasis on the calculation of ultimate shear capacity was studied in this paper. A review of existing design methods and codified provisions was first presented. A total of thirty four experiments carried out on stainless steel plate girders of the austenitic, duplex and lean duplex grades were collected and used to assess the current shear design expressions of EN 1993-1-4, EN 1993-1-5 and those proposed by Estrada et al [3]. It was found that the current EN 1993-1-4 shear design expressions are conservative and better results can be achieved by using the proposed design expressions of Estrada et al. and EN 1993-1-5. However, further improvements were possible and, on the basis of the available structural performance data revised design equations for the calculation of the ultimate shear capacity of stainless steel plate girders have been proposed. The proposed design expressions were developed in a form similar to those of EN 1993-1-4 and EN 1993-1-5 to retain compatibility with current provisions. Revised expressions for the shear buckling reduction factor χ_w for stainless steel plate girders that account for end post rigidity over the full range of web panel aspect ratios were proposed. The proposals were subjected to a statistical analysis in accordance with EN 1990, where the reliability of the design recommendations were verified.

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Figures and Tables

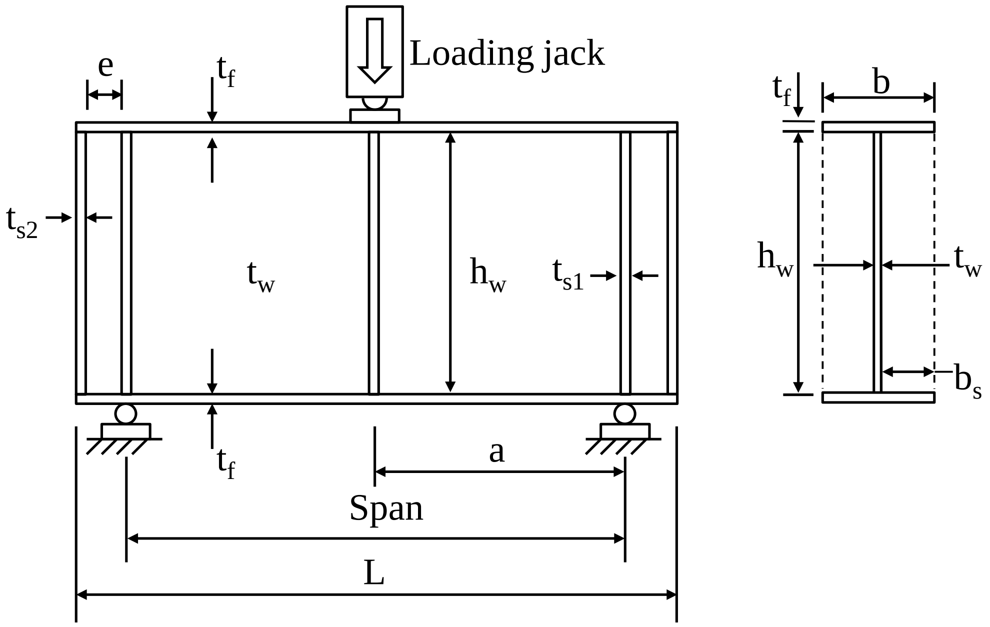

Fig. 1: Geometry of the tested plate girders.

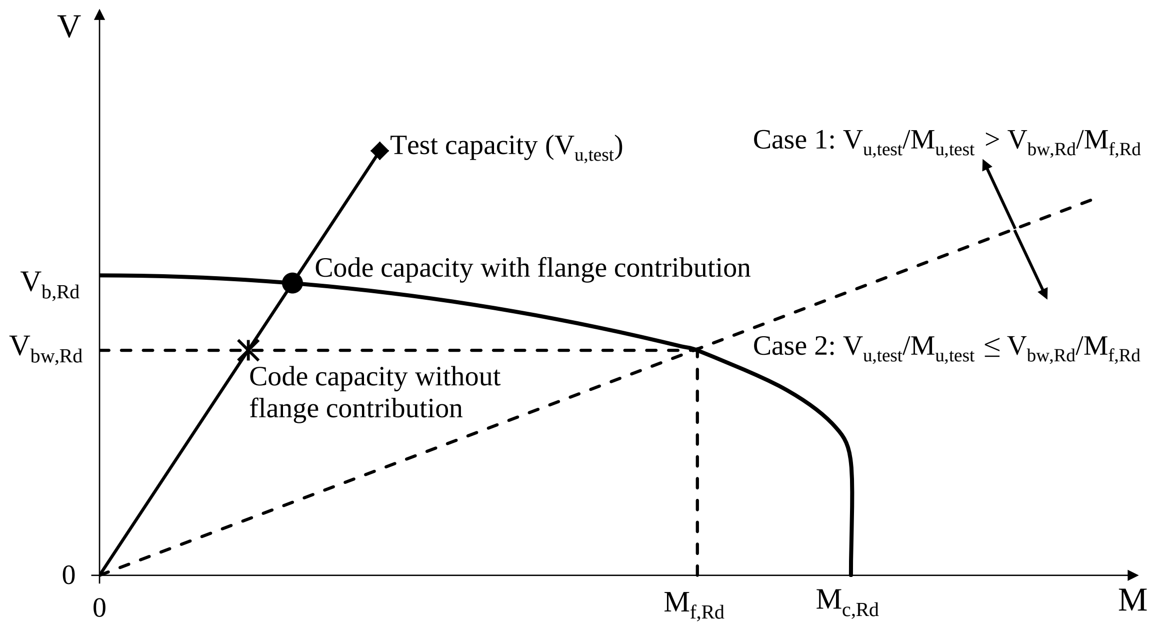

Fig. 2: Moment-shear interaction diagrams and definition of cases 1 and 2.

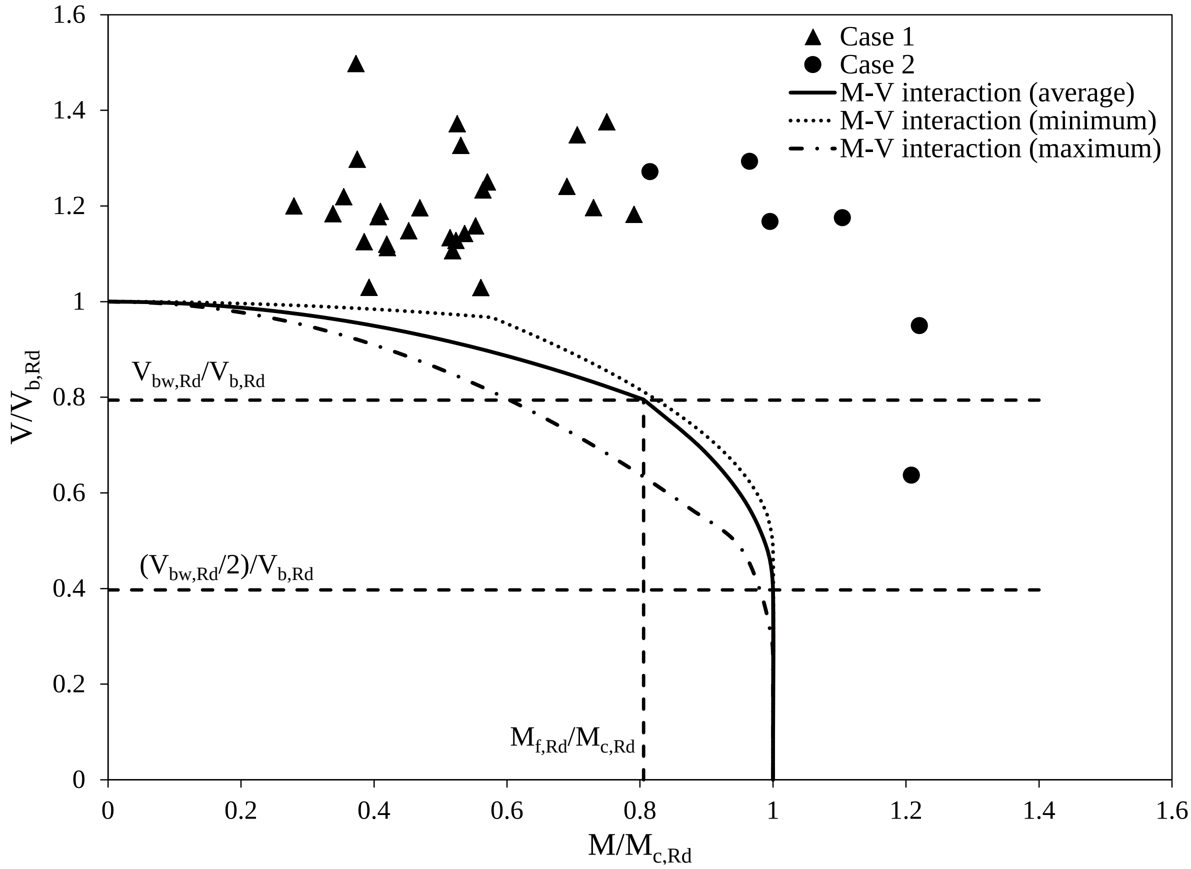

Fig. 3: Test data and normalised moment-shear interaction diagram according to EN 1993-1-4 (2006) [1].

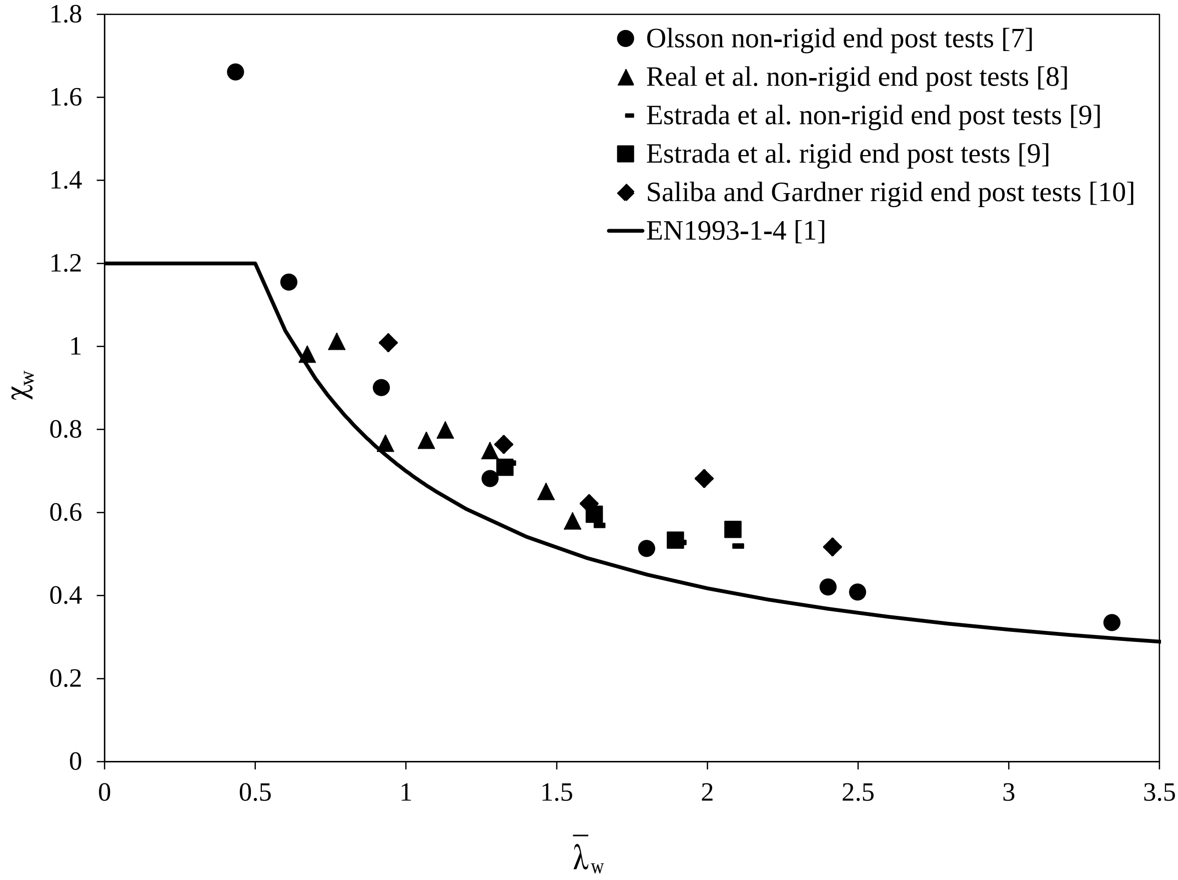

Fig. 4: Comparison between experimental results of case 1 only and the shear resistance function of EN 1993-1-4 (2006).

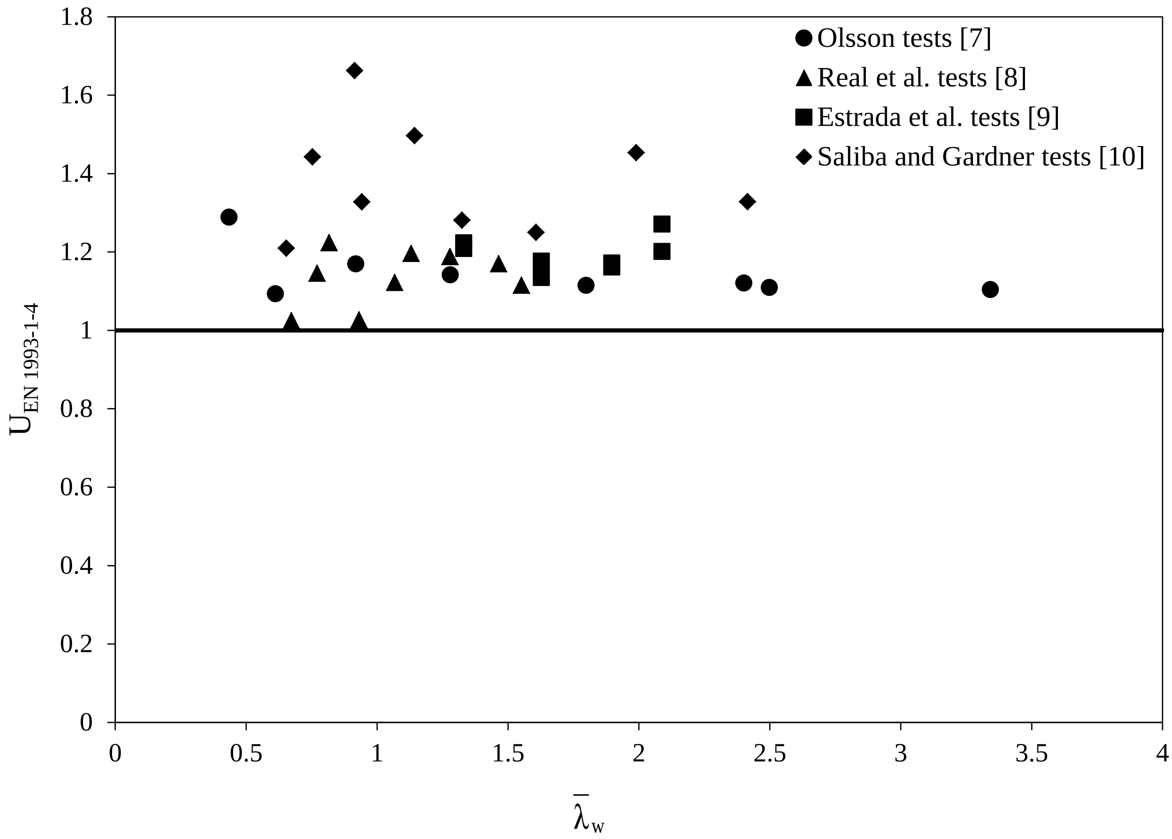

Fig. 5: Utilization ratio (test/design resistance) as obtained from EN 1993-1-4 (2006).

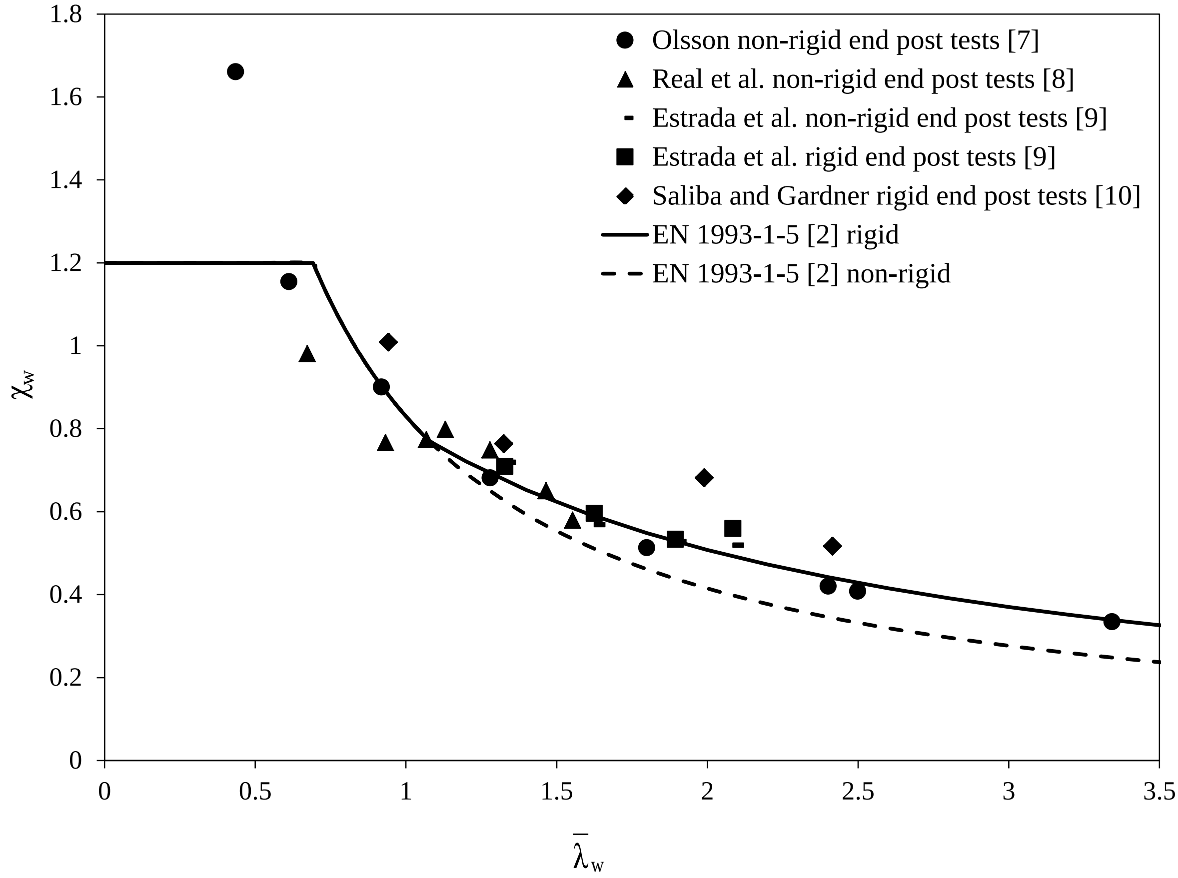

Fig. 6: Comparison between experimental results of case 1 only and the shear resistance function of EN 1993-1-5 (2006) [2].

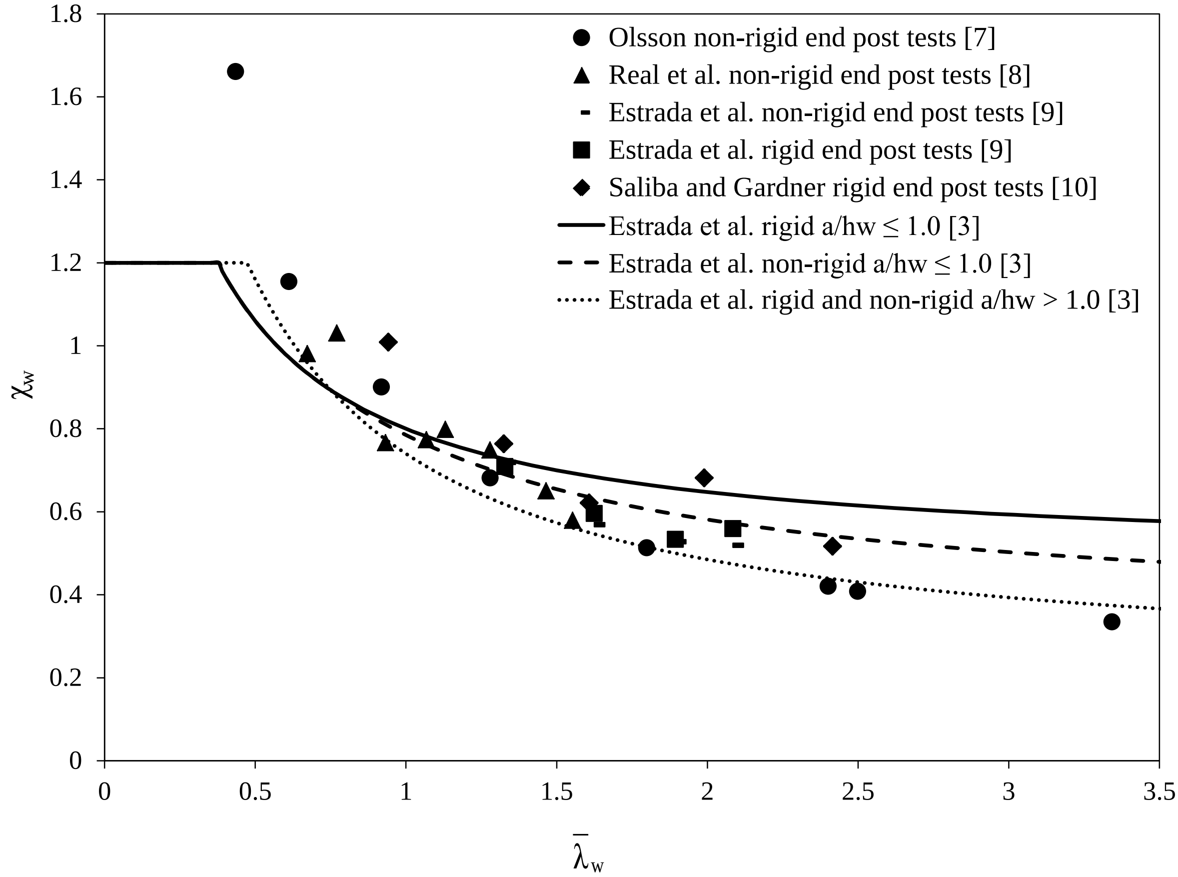

Fig. 7: Comparison between experimental results of case 1 only and the shear resistance function of Estrada et al. [3].

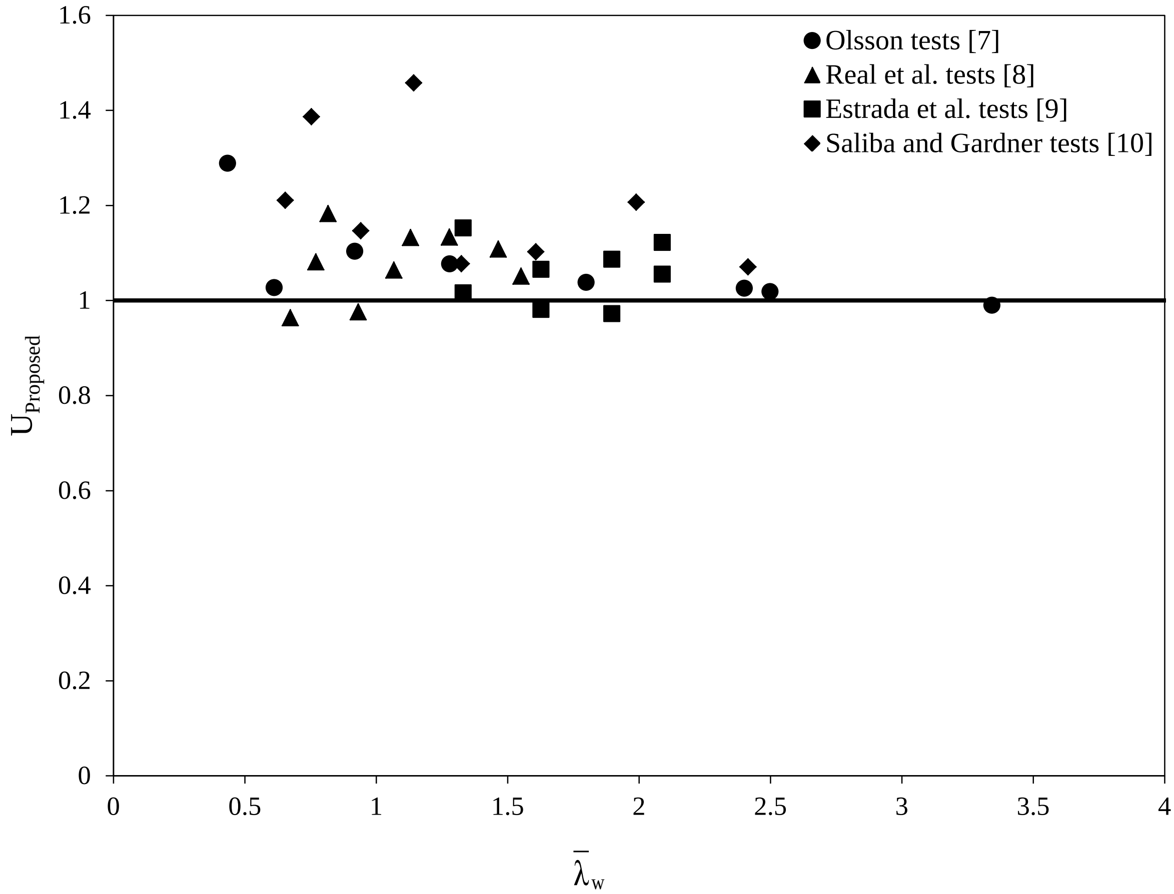

Fig. 8: Utilization ratio (test/design resistance) as obtained from the proposed equations of this paper.

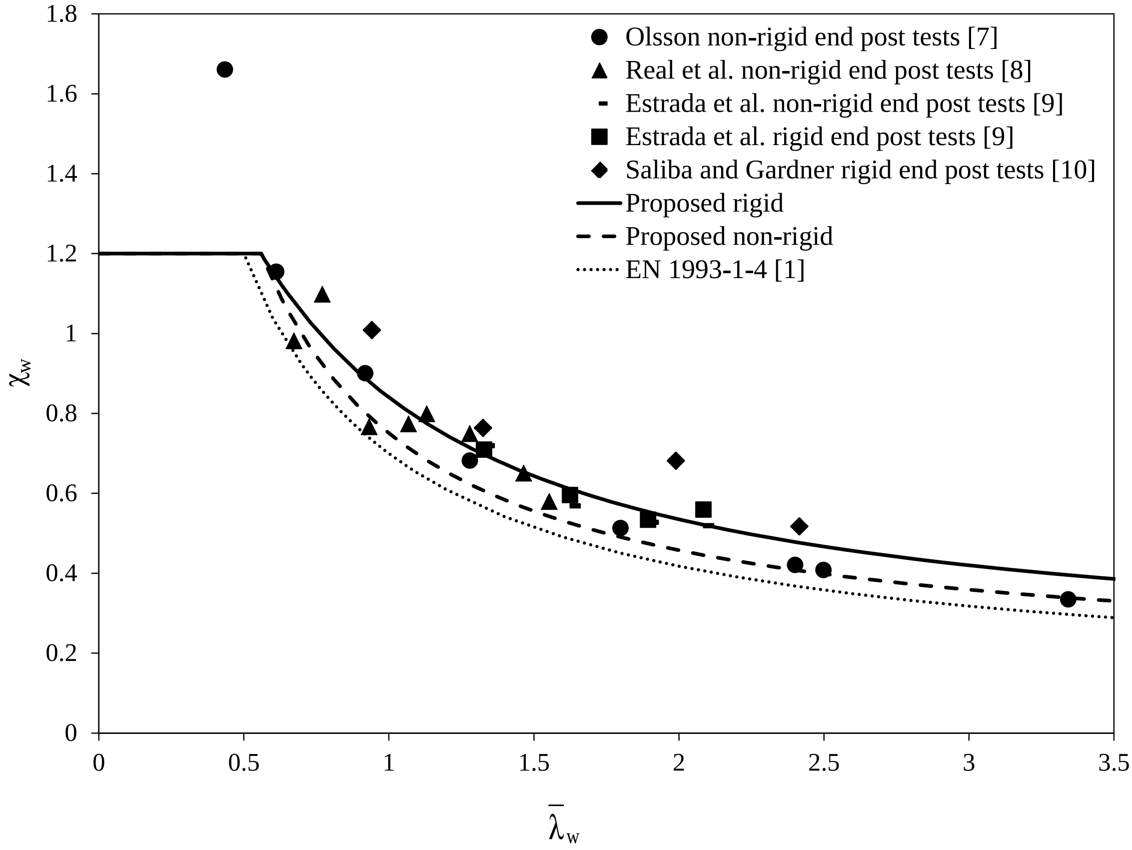

Fig. 9: Comparison between experimental results of case 1 only and the shear resistance functions of EN 1993-1-4 (2006) [1] and the proposed approach.

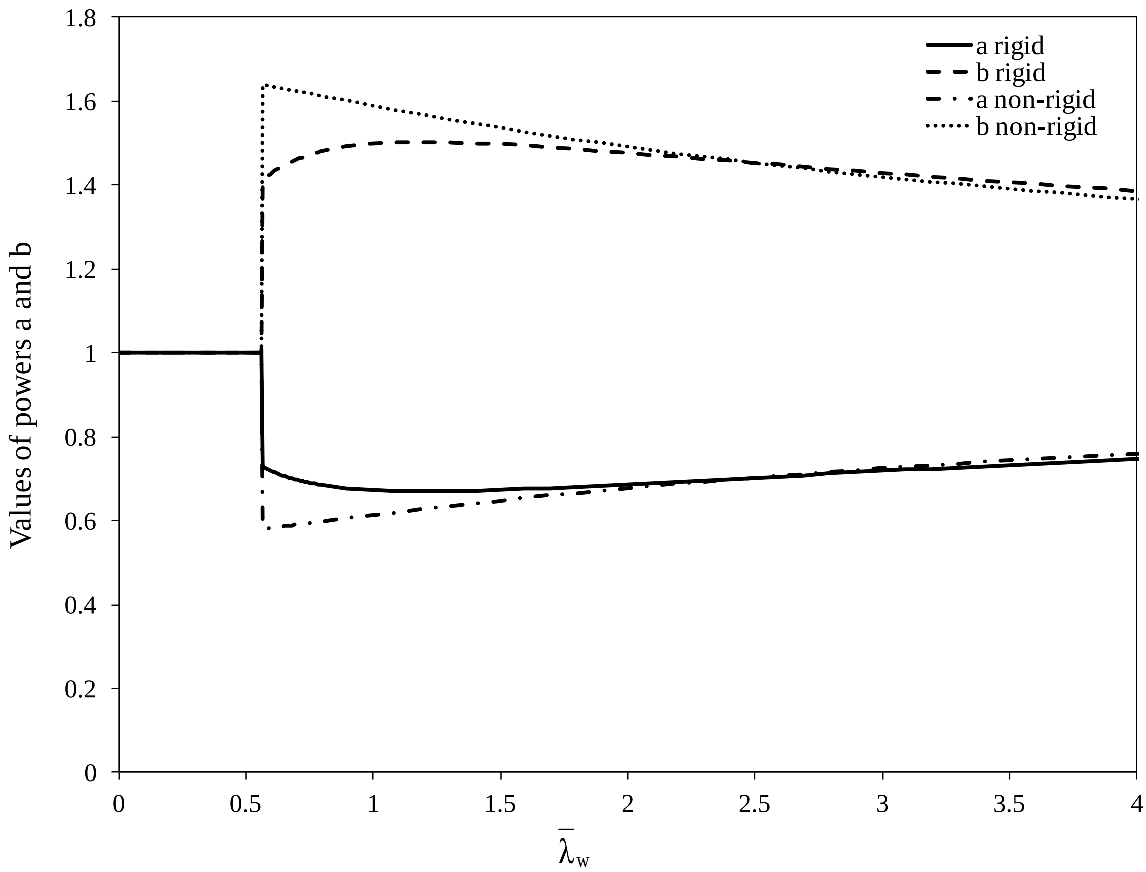

Fig. 10: The powers a and b for rigid and non-rigid end posts versus web slenderness $\bar{\lambda}_w$.

Table 1: Collected experimental data on stainless steel plate girders

End-post	Reference	Label	Grade	$\sigma_{0.2,w}$ (N/mm^2)	$\sigma_{0.2,f}$ (N/mm^2)	L (mm)	a (mm)	h_w (mm)	b (mm)	t_f (mm)	t_w (mm)	t_{s1} (mm)	t_{s2} (mm)	a/h_w	$\bar{\lambda}_w$	$V_{u,test}$ (kN)
Non-rigid	Olsson [7]	NR1	1.4301	297	285	1049	449	146	200	12	4	12	-	3.08	0.44	179
		NR2		297	285	2100	901	297	199	12	4	12	-	3.03	0.92	190
		NR3		297	285	2998	1200	597	200	12	4	12	-	2.01	1.80	226
		NR4		297	285	3997	1600	793	201	12	4	12	-	2.02	2.40	242
		NR5	1.4462	573	525	1051	450	148	200	13	4	13	-	3.04	0.61	269
		NR6		573	525	2100	900	298	200	13	4	13	-	3.02	1.28	295
		NR7		573	525	2996	1200	597	203	13	4	13	-	2.01	2.50	366
		NR8		573	525	3997	1600	795	202	13	4	13	-	2.01	3.34	388
	Real et al. [8]	NR9	1.4301	323	267	1000	500	500	200	20	8	20	-	1.00	0.67	804
		NR10		323	267	1000	500	500	200	20	6	20	-	1.00	0.93	531
		NR11		301	267	1000	500	500	200	20	4	20	-	1.00	1.28	353
		NR12		323	267	1500	750	500	200	20	8	20	-	1.50	0.77	756
		NR13		323	267	1500	750	500	200	20	6	20	-	1.50	1.07	484
		NR14		301	267	1500	750	500	200	20	4	20	-	1.50	1.46	284
		NR15		323	267	2000	1000	500	200	20	8	20	-	2.00	0.82	714
		NR16		323	267	2000	1000	500	200	20	6	20	-	2.00	1.13	467
		NR17		301	267	2000	1000	500	200	20	4	20	-	2.00	1.55	243
	Estrada et al. [9]	NR18		301	267	2360	1050	700	200	20	4	20	-	1.50	2.09	309
		NR19		301	267	2660	1200	600	200	20	4	20	-	2.00	1.90	261
		NR20		301	267	2760	1250	500	200	20	4	20	-	2.50	1.63	228
		NR21		301	267	2860	1300	400	200	20	4	20	-	3.25	1.33	218
Rigid	Estrada et al. [9]	R1	1.4301	301	267	2360	1050	700	200	20	4	20	20	1.50	2.09	327
		R2		301	267	2660	1200	600	200	20	4	20	20	2.00	1.90	263
		R3		301	267	2760	1250	500	200	20	4	20	20	2.50	1.63	237
		R4		301	267	2860	1300	400	200	20	4	20	20	3.25	1.33	215
	Saliba and Gardner [10]	R5	1.4162	513	490	1360	600	600	200	12	4	20	20	1.00	1.99	562
		R6		528	490	1360	600	600	200	12	6	20	20	1.00	1.32	888
		R7		475	490	1360	600	600	200	12	8	20	20	1.00	0.94	1326
		R8		471	490	1360	600	600	200	12	10	20	20	1.00	0.75	1838
		R9		513	490	2560	1200	600	200	12	4	20	20	2.00	2.41	396
		R10		528	490	2560	1200	600	200	12	6	20	20	2.00	1.61	682
		R11		475	490	2560	1200	600	200	12	8	20	20	2.00	1.14	976
		R12		471	490	2560	1200	600	200	12	10	20	20	2.00	0.91	1162
		R13		560	560	2560	1200	600	200	15	15	20	20	2.00	0.65	1801

Table 2: Utilization ratios (test/design resistance) for plate girders as obtained from EN 1993-1-4 (2006), EN 1993-1-5 (2006), Estrada et al. [3] and proposed design methods

Reference	Label	$U_{EN1993\text{-}1\text{-}4}$	$U_{EN1993\text{-}1\text{-}5}$	$U_{Estrada}$	$U_{Proposed}$
Olsson [7]	NR1	1.29	1.31	1.29	1.29
	NR2	1.17	0.99	1.13	1.10
	NR3	1.11	1.11	1.00	1.04
	NR4	1.12	1.20	0.96	1.03
	NR5	1.09	1.00	1.09	1.03
	NR6	1.14	1.02	1.06	1.08
	NR7	1.11	1.20	0.96	1.02
	NR8	1.10	1.31	0.91	0.99
Real et al. [8]	NR9	1.02	0.86	1.04	0.96
	NR10	1.03	0.89	0.96	0.98
	NR11	1.19	1.06	1.05	1.13
	NR12	1.15	0.96	1.12	1.08
	NR13	1.12	0.99	1.07	1.06
	NR14	1.17	1.07	1.09	1.11
	NR15	1.22	1.11	1.21	1.18
	NR16	1.20	1.07	1.13	1.13
	NR17	1.12	1.04	1.02	1.05
Estrada et al. [9]	NR18	1.20	1.23	1.07	1.12
	NR19	1.16	1.14	1.04	1.09
	NR20	1.13	1.07	1.03	1.07
	NR21	1.22	1.09	1.13	1.15
Estrada et al. [9]	R1	1.27	1.11	1.13	1.06
	R2	1.17	1.00	1.05	0.97
	R3	1.18	0.99	1.07	0.98
	R4	1.21	1.02	1.12	1.02
Saliba and Gardner [10]	R5	1.45	1.29	1.04	1.21
	R6	1.28	1.12	1.04	1.08
	R7	1.33	1.14	1.21	1.15
	R8	1.44	1.36	1.43	1.39
	R9	1.33	1.16	1.15	1.07
	R10	1.25	1.08	1.13	1.10
	R11	1.50	1.46	1.48	1.46
	R12	1.66	1.66	1.66	1.66
	R13	1.21	1.22	1.21	1.21
Mean		1.22	1.13	1.12	1.12
COV		0.11	0.14	0.14	0.13

Table 3: Proposed design expressions for the calculation of the web contribution to the shear resistance

	χ_w for rigid end post	χ_w for non-rigid end post
$\bar{\lambda}_w \leq 0.56$	1.2	1.2
$\bar{\lambda}_w > 0.56$	$0.17 + 0.79/\bar{\lambda}_w - 0.12/\bar{\lambda}_w^2$	$0.16 + 0.60/\bar{\lambda}_w - 0.01/\bar{\lambda}_w^2$

Table 4: Results of statistical analysis of test data for proposed web shear resistance equations

Data set	No. of tests n	Fractile factor $k_{d,n}$	$R_{test}/R_{proposed}$ b	Model scatter V_δ	Resistance scatter V_r	γ_{M1}
Austenitic tests only	20	3.32	1.05	0.060	0.084	1.04
Lean duplex tests + FE [10]	25	3.32	1.11	0.081	0.093	1.06