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Experimental study of the shear response of lean duplex stainless steel plate girders

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Abstract

An experimental and numerical study of the shear response of lean duplex stainless steel plate girders is described in this paper. A total of nine welded plate girders were tested. Lean duplex stainless steel, which is a low nickel variety of the material, has approximately twice the strength of the common austenitic grades and at approximately half the initial cost. It also possesses good corrosion resistance and high temperature properties, as well as adequate weldability and fracture toughness. Two web panel aspect ratios and a range of web slendernesses were considered. The results from the experiments, including the full load-deformation histories and failure models are reported. A numerical investigation was carried out in parallel with the testing. The models were first validated against the experimental results after which parametric studies were performed to generate data for a wider range of cross-sections. The generated experimental and numerical data were used to assess the shear resistance design equations given in Eurocode 3: Part 1.4. The current design provisions were shown to be safely applicable to lean duplex stainless steel, though improved guidance is sought in further ongoing research.

Keywords: Experiments, Failure modes, Finite element, Lean duplex, Numerical modelling, Shear buckling, Stainless steel, Structural testing, Ultimate shear capacity, Welded I-sections,.

1. Introduction

Stainless steel has a range of physical characteristics such as high strength, stiffness, and ductility together with excellent durability that makes it ideally suited to use in construction [1]. However, due to the high initial cost of structural stainless steel compared to that of structural carbon steel, and the relatively limited awareness of its credentials among structural engineers, the use of stainless steel in construction remains rather restricted. The high initial cost of stainless steel is largely attributed to the nickel content which, for the commonly used grades of austenitic stainless steel, lies between about 8% and 10% by mass. A new breed of stainless steel has been developed with low nickel content (around 1.5%), which may offer improved economy over existing grades. The material, referred to as lean duplex stainless steel, possesses higher strength than the austenitic grades, is less expensive, retains good corrosion resistance and high temperature properties, and has adequate weldability and fracture toughness [2-4].

Although lean duplex stainless steel has already featured in the construction of a number of footbridges, and is included in the material standard EN 10088-4 [5], it is not currently covered by any structural design code, due to its structural characteristics having been largely unverified. In order to overcome this, a comprehensive programme of experimental and numerical research is underway at Imperial College London to investigate the structural performance of lean duplex stainless steel. Initial research concentrated on cold-formed hollow sections [6, 7], while more recently the focus has moved to welded I-sections [8] and, in the present paper, plate girders. Other recent studies of cold-formed lean duplex stainless steel hollow sections have been performed by Huang and Young [9] and Patton and Singh [10].

The present paper examines the behaviour of lean duplex stainless steel welded plate girders through experimentation and finite element (FE) modelling. Unlike in the case of carbon steel, the shear behaviour of stainless steel plate girders has only received

relatively limited attention, and no previous experiments have been conducted on plate girders fabricated from lean duplex material. Structural testing has been carried out on austenitic and duplex stainless steel plate girders by Olsson [11], Real et al. [12], and Estrada et al. [13], while recent numerical studies have been undertaken by Hassanein [14]. Structural design rules for stainless steel plate girders are relatively scarce; the provisions of Eurocode 3: Part 1.4 [15], and their applicability to lean duplex stainless steel are examined later in this paper. First, structural performance data are generated through laboratory testing and numerical modelling, as described in Sections 2 and 3 of this paper.

2. Experimental programme

An experimental programme, comprising material tests, measurements of geometric imperfections and plate girder tests, is described in this section.

2.1. Material testing

A series of tests on tensile and compressive coupons extracted from the plate material used to form the structural cross-sections was carried out in the Structures Laboratory of the Department of the Civil and Environmental Engineering at Imperial College London. The coupon tests were used to obtain the basic stress-strain properties and to analyze the nonlinear material behaviour of lean duplex stainless steel. A total of twenty-eight coupons of seven material thicknesses (4 mm, 6 mm, 8 mm, 10 mm, 12 mm, 15 mm, and 20 mm) were tested in tension and compression. For each thickness, two tensile coupons and two compressive coupons were tested. Tests were performed on both longitudinal (i.e. parallel to the rolling direction) and transverse (i.e. perpendicular to the rolling direction) coupons, following the provisions of EN 10002-1 [16]. The nominal dimensions of the necked tensile coupons were 340×30 mm while the nominal dimensions of the compressive coupons were 72×16 mm. For the thicker coupons (15 mm and 20 mm) the tests were carried out in an INSTRON 600 kN machine, whereas for the remaining coupons the tests were carried out in an INSTRON 250 kN machine. The initial strain rate used for all of the material tests was 0.0003s^{-1} , increasing after 1% strain to a strain rate of 0.008s^{-1} . The tensile coupons were marked

with proportional gauge lengths to calculate their plastic strain at fracture ϵ_f after testing, based on elongation over the standard gauge length $5.65\sqrt{A_c}$, where A_c is the cross-sectional area of the coupon. Tensile strain was measured by means of a digital extensometer. The compressive coupons were tested in a bracing jig to prevent buckling; strain gauges were attached to both sides of the tested coupons at mid-height. All test data, including load, displacement, strain and input voltage were recorded at one-second intervals using the data acquisition system DATASCAN. The obtained material data for each of the tensile and compressive test specimens, together with the mill certificate material properties provided by the manufacturer, are given in Tables 1 and 2, where E is Young's modulus, $\sigma_{0.2}$ is the 0.2% proof stress, $\sigma_{1.0}$ is the 1.0% proof stress, σ_u is the ultimate tensile stress, ϵ_f is the plastic strain at fracture based on elongation over the standard gauge length and n and $n'_{0.2,1.0}$ are strain hardening exponents for the compound Ramberg-Osgood model [17, 18] as modified by Gardner and Ashraf [19]. The stress-strain curves for each of the tested thicknesses are presented in Figs 1-7.

A comparison between the results obtained from the measured tensile and compressive material tests for the 0.2% and 1% proof stress shows that generally the results of the longitudinal coupons are lower than those of the transverse coupons. Furthermore, the measured 0.2% proof stress from the transverse coupons tests were found to be relatively similar to those stated in the mill certificates, whereas the longitudinal coupons were found to be generally lower. This is consistent with the fact that mill tests are performed in the transverse direction. Strain rates are also typically higher in mill tests than laboratory testing programmes. The mean measured values of the Ramberg-Osgood n and $n'_{0.2,1.0}$ parameters for the tested plate material were 10.5 and 2.6 respectively with some variation depending on rolling (longitudinal or transverse) and loading (tension or compression) directions. The obtained values are close to those reported by Saliba and Gardner [8] ($n = 10.0$ and $n'_{0.2,1.0} = 2.0$) following coupon tests on similar lean duplex stainless steel plate material used to form welded I-sections. The values reported by Theofanous and Gardner [6, 7] from tests on cold-formed lean duplex stainless steel hollow section material ($n = 7.0$ and $n'_{0.2,1.0} = 3.3$), indicate a more rounded stress-strain response, which is typical of cold-formed material.

2.2. Plate girder tests

The material examined in the previous section was used to fabricate nine lean duplex stainless steel plate girders. The fillet welds between the flanges and web were made using submerged arc welding with weld material 22 9 3 LN, designated according to EN 12072 [20]. The stiffeners were MIG welded using the same weld material. Prior to testing, the basic geometry of the plate girders was carefully measured. The key measurements can be found in Table 3 where, with reference to Fig. 8, L is the specimen length, a is the web panel length, e is the distance between the end post and the internal stiffener over the support, b is the overall flange width, h_w is the web depth, t_f is the thickness of the flanges, t_w is the thickness of the web, $b_s = (b - t_w)/2$ is the width of the stiffeners and t_s is the thickness of the stiffeners. Local geometric imperfections were not analysed in detail but approximate sample measurements found imperfection amplitudes to be similar to those observed in lean duplex stainless steel welded I-sections tested previously by Saliba and Gardner [8], which were fabricated in the same manner.

A total of eighteen strain gauges were attached to each of the plate girders at critical points, and were used to monitor the development of the shear buckling phenomenon during the tests and to validate the numerical models. All sections had Class 4 webs in bending according to EN 1993-1-4 [15], with the exception of Specimen I-600×200×15×15-2, for which the web was Class 3. Rigid end post conditions, satisfying the requirements of EN 1993-1-5 [21] for carbon steel plate girders, were achieved through the provision of a pair of transverse web stiffeners at each support. The tests were carried out to generate data in order to assess the applicability of the current design methods available for stainless steel plate girders to the lean duplex grades and to understand further the general behaviour of stainless steel plate girders failing in shear.

Two sets of specimens were tested – the first set had four specimens with a web panel aspect ratio $a/h_w = 1.0$, while the second set had five specimens with a web panel aspect ratio $a/h_w = 2.0$. The plate girders with web panels of aspect ratio 1.0 were 1360 mm in length, whereas those of aspect ratio 2.0 were 2560 mm in length. For each of the tested plate girders, instrumentation was provided in one panel only, while shear buckling was

inhibited in the adjacent panel through the addition of wooden diagonal planks. A total of eighteen strain gauges were attached to each of the plate girders at critical points to monitor the development of the shear buckling phenomenon during the tests. Ten triaxial (rosette) strain gauges were attached to the web panel – five on each side, to measure the membrane and bending components of strains in the web. Three of the five triaxial strain gauges were placed along the central vertical axis of the web panel. These strain gauges were used to study the development of the shear stress state and the inclination of the tension band once shear buckling occurs. The other two triaxial strain gauges were located at the corner of the web panel to examine anchorage of the tension field. Eight uniaxial strain gauges were attached to the flanges and stiffeners; the strain gauges in the flanges were used to monitor the formation of plastic hinges. The locations of each of the above mentioned strain gauges are shown in Fig. 9, in which g defines the positions of the strain gauges and c is the calculated distance from the stiffener to the plastic hinge location, determined according to EN 1993-1-4 [15]. Furthermore, for each of the tested plate girders, six Linear Variable Displacement Transducers (LVDTs) were employed to measure the vertical and horizontal displacements. Three LVDTs were used to measure vertical displacements – one at mid-span and one at each of the two supports. The other three LVDTs were located horizontally along the mid-height of the web panel to measure the out-of-plane lateral displacements associated with the shear buckling. Two string potentiometers were employed to measure the change in length of the two diagonals of the web panels, and hence to estimate the average shear strain. Finally, two inclinometers were placed at each end of the beam in order to determine the rotation at the support points. The general scheme of the LVDTs, string potentiometers and inclinometers can be found in Fig. 10.

The tests were carried out in an ENERPAC 4000 kN machine controlled by a Walter + Bai ag DIGICON 2000 digital controller. The tests were load-controlled at a rate of 10 kN/sec, and by regulating the flow rate into the hydraulic jack, the unloading response could be captured. The load was applied through a bearing plate, which was 200 mm wide (equal to the flange width), 100 mm in length and 50 mm thick. Loads, rotations, displacements and strains were all recorded at one-second intervals using the data acquisition system DATASCAN. The employed specimen designation system begins

with the section type, followed by section depth $\times$ flange width $\times$ flange thickness $\times$ web thickness, and finally the web panel aspect ratio (1 or 2).

The key experimental results from the plate girder tests are summarised in Table 3, where V_{cr} is the calculated elastic critical shear buckling load of the web, $V_{u,test}$ and $M_{u,test}$ are the shear force and bending moment, respectively, at failure from the experiments and c_{test} is the measured horizontal dimension from the support stiffener to the plastic hinge that formed in the flange at collapse. The observed failure modes, which are discussed further in Section 4.2, are also included in Table 3. The obtained load-displacement curves from the two series of tests – $a/h_w = 1.0$ and $a/h_w = 2.0$ – are shown in Figs 11-14. Figs 11 and 12 show load versus mid-span vertical deflection, while Figs 13 and 14 show load versus lateral deflection of the web. The results of the experiments are employed for the validation of the numerical models in Section 3 of this paper and are analysed and discussed in Section 4, where typical failure modes are also shown.

3. Numerical modelling

3.1. Basic modelling assumptions

The experiments described in the previous section were supplemented by a numerical study using the general-purpose finite element analysis package ABAQUS [22]. The experiments were initially used to validate the FE models. Following the successful validation of the numerical models, parametric studies were carried out to assess the influence of key parameters on the structural response of lean duplex stainless steel plate girders. Furthermore, the parametric studies were used to expand the available structural performance data over a wider range of web slendernesses and to consider different end post conditions.

The finite element type used for the numerical analyses was the four-noded doubly curved general-purpose shell element S4R with finite membrane strains. This element has been shown to be suitable for the modelling of a range of shell thicknesses and has

been successfully employed in the modelling of lean duplex stainless steel welded I-sections [8] and other similar structural components [6, 7, 23, 24].

The FE models were initially developed based on the measured geometry of the tested plate girders. For all of the FE models reported in this paper the stress-strain response of the lean duplex stainless steel was represented initially by means of the two-stage Ramberg-Osgood model described by Gardner and Ashraf [19]. For incorporation into ABAQUS, the continuous curve was approximated with a multi-linear model defined in terms of true stress and log plastic strain. The relationship between true stress and engineering stress, σ_{true} and σ_{nom} , respectively, is given by Eq. (1), while the relationship between log plastic strain and engineering strain, $\epsilon_{\text{ln}}^{\text{pl}}$ and ϵ_{nom} , respectively, is given by Eq. (2), where σ_{nom} and ϵ_{nom} are the engineering stress and strain respectively and E is the Young's modulus. The material properties incorporated into the FE models were based on the average of the longitudinal and transverse coupon test results, with tensile material properties applied to the lower halves of the models and compressive material properties applied to the upper halves.

$$\sigma_{\text{true}} = \sigma_{\text{nom}} (1 + \epsilon_{\text{nom}}) \quad (1)$$

$$\epsilon_{\text{ln}}^{\text{pl}} = \ln(1 + \epsilon_{\text{nom}}) - \frac{\sigma_{\text{true}}}{E} \quad (2)$$

Boundary conditions were carefully incorporated into the FE models to replicate the experimental setup. The load was applied at the centre of the FE models through a bearing plate (similar to the experimental setup; see Section 2.2 and Fig. 8). Vertical and lateral displacements were restrained at the supports, while longitudinal displacements and rotation about the horizontal axes were restrained at mid-span.

Initial geometric imperfections, arising principally during production and fabrication, are present in all structural members and influence their structural response. Therefore, it is necessary to account for these imperfections in any numerical modelling study. This is commonly achieved by first performing a linear eigenvalue buckling analysis to determine the lowest relevant elastic buckling mode, and subsequently incorporating the obtained shape as the geometric imperfection, with a chosen amplitude, in the

nonlinear analysis; this was the approach adopted in the present study. Owing to the absence of global (member) buckling, only local geometric imperfections were incorporated in the FE models. The modified Riks method [22], which is an algorithm that enables effective solutions to be found to unstable problems and adequately traces nonlinear unloading paths, was used in the nonlinear analyses. The imperfection amplitude employed in this study was the one derived from the Dawson and Walker model [25], as modified by Gardner and Nethercot [26], defined by Eq. (3), where $\sigma_{0.2}$ is the tensile 0.2% proof stress and σ_{cr} is the elastic critical buckling stress of the most slender constituent plate element in the section, determined on the basis of the flat width of the element. This imperfection amplitude prediction model has been successfully employed for the modelling of lean duplex stainless steel welded I-sections in compression and bending [8], and further similar applications [6, 7, 27]. The imperfection amplitude $\omega_{D\&W}$ is given by Eq. (3) and presented in Table 4.

$$\omega_{D\&W}=0.023(\sigma_{0.2}/\sigma_{cr})t \quad (3)$$

The study by Saliba and Gardner [8] accounted for residual stresses in the modelling of the lean duplex stainless steel welded I-sections but found insignificant variation in structural behaviour when models with and without residual stresses were compared. It was therefore concluded that residual stresses need not be included in the models, and the same approach is taken herein.

3.2. Validation of models

The accuracy of the numerical models and their suitability for performing parametric studies was initially assessed by comparing the experimental results to those generated by the FE simulations. The comparisons were based on the general load-displacement curves and failure modes, as well as other key values such as the ultimate shear force, denoted $V_{u,test}$ and $V_{u,FE}$ for the experimental and numerical results, respectively. Note that for the present loading configuration, maximum shear force is simply equal to half the applied load. The ratio of the numerical to experimental ultimate shear forces ($V_{u,FE}/V_{u,test}$) is presented in Table 4. The ultimate shear force is generally very well predicted with the Dawson and Walker imperfection amplitude model, with a mean value of $V_{u,FE}/V_{u,test}$ of 1.00 and a coefficient of variation (COV) of 0.03. The failure

modes obtained from the numerical models showed good agreement with those obtained from the experiments. Typical FE failure modes for the plate girders of aspect ratios 1.0 and 2.0 are shown in Figs 15 and 16, respectively. The general shapes of the load-deflection curves from the FE models also closely matched those obtained from experiments. A comparison between the experimental and numerical load-displacements curves for specimens I-600×200×12×8-1 and I-600×200×12×8-2 are depicted in Figs 17 and 18, respectively.

The principal strains obtained from the rosette strain gauges at the centre and corner of the tested web panels were also compared with the values extracted from the corresponding locations in the FE models; close agreement was observed, as shown for specimen I-600×200×12×8-1 in Figs 19 and 20. In both cases, yielding of the tension field and the subsequent loss of anchorage may be seen to correspond to the attainment of the peak load. A further comparison between the experimental and numerical results is shown in Fig. 21, where the inclination of the principal plane to the horizontal is shown for specimen I-600×200×12×4-1. The following progression of behaviour may be observed from both the test and FE results: At the early stage of loading, the web panel is in the state of pure shear (i.e. the principal plane is inclined at 45° to the horizontal); once the applied shear force exceeds the elastic shear buckling load, stresses along the tensile diagonal continue to increase while those along the compressive diagonal develop at a reduced rate, and hence the principal plane begins to rotate towards the horizontal; eventually the web panel yields under the combined stress field and an ultimate capacity is reached.

3.3. Parametric studies

Having achieved good overall agreement between the numerical and experimental results, a series of parametric studies were initiated to expand the available results over a wider range of plate girder web slendernesses and aspect ratios and to consider different end post conditions. The experimental results, together with the parametric study results, are then used to assess the general structural behaviour of lean duplex stainless steel plate girders and to investigate the applicability of the current codified design provisions. The generated models follow the basic assumptions stated in Section 3.1. The parametric studies covered three types of plate girders: (1) rigid end post plate girders of constant length, web height, flange width and flange thickness (similar to

those employed in the experiments) but with varying web thickness to create a range of web slendernesses $\bar{\lambda}_w$ between 0.6 and 5.0, (2) rigid end post plate girders similar to (1) but with modified geometries to examine stocky cross-sections with web slendernesses ranging between 0.3 and 0.5, and (3) non-rigid end post plate girders with the same cross-sectional dimensions as employed in case (1). The material properties adopted in the FE parametric studies were based on the average of the experimental material stress-strain curves from the seven tested plate thicknesses, which were $E = 206000 \text{ N/mm}^2$, $\sigma_{0.2} = 500 \text{ N/mm}^2$, $\sigma_{1.0} = 580 \text{ N/mm}^2$, $n = 10$ and $n'_{0.2,1.0} = 2.3$.

4. Analysis of results

4.1. Introduction

In this section, the results from the laboratory tests and the numerical analyses are examined and used to assess the applicability of the shear resistance design model given in Eurocode 3: Part 1.4 [15] to lean duplex stainless steel. Comparisons of the structural performance of lean duplex stainless steel plate girders with that of the more common austenitic and ferritic stainless steel grades are also made. Note that all comparisons have been carried out using the measured geometry and material properties, and with all partial factors set to unity.

4.2. Failure modes

Three different failure modes were identified from the experiments: (1) a shear dominant failure featuring shear buckling of the plate girder web, (2) a bending dominant failure featuring local buckling of the plate girder compression flange and (3) a combined bending plus shear failure, involving an interaction of failure modes (1) and (2). These three observed failure modes are discussed below.

4.2.1. Shear dominant failure

A shear dominant mode of failure was observed in test specimens I-600×200×12×4-1, I-600×200×12×6-1, I-600×200×12×4-2 and I-600×200×12×6-2 and many of the

numerically generated results. A typical example of this mode of failure, obtained from plate girder I-600×200×12×4-1, is shown in Fig. 22. The moment-shear interaction diagram for this specimen, determined according to EN 1993-1-4 [15], is shown in Fig. 23, where V is the applied shear force, $V_{b,Rd}$ is the shear buckling resistance of the cross-section including the flange contribution, $V_{bw,Rd}$ is the shear buckling resistance of the web only, M is the applied moment, $M_{c,Rd}$ is the cross-section bending resistance, $M_{f,Rd}$ is the bending resistance of the flanges alone and for Class 4 cross-sections, $M_{eff,Rd}$ is the bending resistance of the effective section. The location of the experimental data point (i.e. a high value of $V/V_{b,Rd}$ and a low value of $M/M_{c,Rd}$) clearly indicates that a shear dominant mode of failure would be anticipated.

4.2.2. Bending dominant failure

The second failure mode observed in the experimental and numerical analyses was a bending dominant mode involving local buckling of the compression flange. This mode of failure was observed in test specimens I-600×200×12×10-1, I-600×200×12×10-2 and I-600×200×15×15-2 and a number of the numerically generated results. A typical example, obtained for specimen I-600×200×12×10-2, is presented in Fig. 24. The moment-shear interaction diagram from EN 1993-1-4 [15] for this plate girder (see Fig. 25) reveals a high value of $M/M_{c,Rd}$ and lower value of $V/V_{b,Rd}$, and hence anticipates a bending dominant failure.

4.2.3. Combined bending plus shear failure

An intermediate failure mode involving an interaction of shear and bending and a combination of the features described in Sections 4.2.1 and 4.2.2 was observed in test specimens I-600×200×12×8-1 and I-600×200×12×8-2 and some of the numerical models. This mode of failure is shown in Fig. 26 for plate girder I-600×200×12×8-1 where both flange and web deformation may be observed. The moment-shear interaction diagram from EN 1993-1-4 [15] for this specimen is given in Fig. 27 where the shear and moment utilisations indicate that a combined shear plus bending failure mode would be expected.

4.3. Eurocode 3: Part 1.4 design method

The design approach adopted in the European pre-Standard ENV 1993-1-4 (2006) [28] for determining the shear resistance of stainless steel plate girders was based on the simple post critical method of ENV 1993-1-1 (1992) [29] which was developed by Dubas [30] based on Höglund's [31, 32] rotated stress field theory ignoring any flange contribution. The development of the shear buckling provisions given in ENV 1993-1-4 [28] considered the only experiments featuring shear buckling of stainless steel sections that were available at the time, which were those of Carvalho et al. [33].

The simple post critical method was later found by Olsson [11] to be rather conservative, and following an experimental and numerical study of a series of stainless steel plate girders, an improved design approach was proposed. Olsson's [11] proposal was based on Höglund's [31, 32] rotated stress field method taking into consideration both the web and the flange contributions to the resistance to shear buckling, but ignoring the effect of the end post; this approach was adopted in the final version of EN 1993-1-4 (2006) [15]. The ultimate shear buckling resistance $V_{b,Rd}$ of stainless steel plate girders according to EN 1993-1-4 [15] is expressed similarly to that of carbon steel plate girders in EN 1993-1-5 [21], as given by Eq. (4), but with modified web and flange contributions, $V_{bw,Rd}$ and $V_{bf,Rd}$, respectively.

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (4)$$

where η is equal to 1.2, f_{yw} is the yield strength of the web, h_w is the depth of the web, t_w is the thickness of the web and γ_{M1} is the shear partial factor. The web contribution is given by:

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (5)$$

in which χ_w is the reduction factor for web buckling, and the flange contribution is given by:

$$V_{bf,Rd} = \left(\frac{b_f t_f^2 f_{yf}}{c \gamma_{M1}} \right) \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right) \quad (6)$$

with the distance c given by Eq. (7):

$$c = \left(0.17 + \frac{3.5 b_f t_f^2 f_{yf}}{t_w h_w^2 f_{yw}} \right) a \quad \text{with} \quad \frac{c}{a} \leq 0.65 \quad (7)$$

where b_f is the flange width, t_f is the flange thickness, f_{yf} is the yield strength of the flange, M_{Ed} is the design bending moment, $M_{f,Rd}$ is the design plastic moment of resistance of a cross-section consisting of the flanges only and a is the shear panel length.

4.4. Comparison of test and numerical shear capacities with EN 1993-1-4 for plate girders with rigid end posts

A comparison between the ultimate capacity achieved by the test specimens, all of which had rigid end posts, and the predicted capacity according to EN 1993-1-4 [15], assuming proportional loading (i.e. the ratio of shear force to bending moment remains constant), is presented in Table 5. In the EN 1993-1-4 [15] calculation, due allowance for the reduced flanged contribution to the shear resistance owing to the coexistent bending moment has been made in accordance with Eq. (6). The comparisons show that the test results consistently lie above the codified predictions, with a mean ratio of test to EN 1993-1-4 capacity of 1.38 and a coefficient of variation (COV) of 0.07. A similar trend may be seen when the parametric FE results for the plate girders with rigid end posts are compared to the codified predictions in Table 6, with a mean ratio of test to EN 1993-1-4 capacity of 1.35 and a COV of 0.15.

For specimens exhibiting a shear dominant failure mode, defined as those where the ratio of shear force to bending moment in the test $V_{u,test}/M_{u,test} > V_{bw,Rd}/M_{f,Rd}$ (see Fig. 28), a comparison between the test and FE results is also plotted with the EN 1993-1-4 [15] design model in Fig. 29. This comparison is presented in terms of shear buckling reduction factor χ_w versus web slenderness $\bar{\lambda}_w$. In locating the test and FE data points, the flange contribution calculated according to Eq. (6), 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.

4.5. Comparison of test and numerical shear capacities with EN 1993-1-4 for plate girders with non-rigid end posts

The current shear design provisions in EN 1993-1-4 [15] do not differentiate between plate girders with rigid and non-rigid end posts. For carbon steel plate girders, EN 1993-1-5 [21] does distinguish between rigid and non-rigid end posts for web slenderness values $\bar{\lambda}_w > 1.08$ and Estrada et al. [34] reported a clear distinction in response between the two cases, particularly for higher web slendernesses and low panel aspect ratios.

Following the successful validation of the numerical models against the test results reported in this paper on plate girders with rigid end posts, parametric studies on plate girders with non-rigid end posts were performed. The results are shown in Fig. 29 and in Table 7, where a mean ratio of test to EN 1993-1-4 capacity of 1.30 and a COV of 0.09 was found. A comparison of the ultimate shear resistances obtained from the FE models with rigid and non-rigid end posts (but otherwise similar geometry) is presented in Table 8, where a mean $V_{u,FE}/V_{u,FE,non-rigid}$ of 1.05 and a COV of 0.05 was found. An increase in shear resistance up to 18% may be observed when rigid end posts are employed. As observed by Estrada et al. [34], the differences are greatest for higher web slendernesses and lower web panel aspect ratios.

4.6. Comparison with other stainless steel grades

The results obtained from the tests carried out in the present study on lean duplex stainless plate girders are compared to existing test results from Olsson [11], Real et al. [12] and Estrada et al. [13] on austenitic and duplex stainless steel plate girders in Fig. 30. Upon examination of the results, the following conclusions can be drawn: firstly, for similar values of slenderness the results from the lean duplex plate girders exhibit similar normalized performance to equivalent plate girders of other stainless steel grades; secondly, the collected test data show that the Eurocode 3: Part 1.4 [15] shear resistance design model can be safely applied to all stainless steel grades, but improved efficiency could be achieved by re-examining the design model and differentiating between rigid and non-rigid end posts.

5. Summary and conclusions

An experimental study into the structural performance of lean duplex stainless steel plate girders has been reported in this paper. The results of the experimental investigation were supplemented by means of a parallel numerical study. The numerical models were initially validated against the experimental results and subsequently used to conduct parametric studies to investigate a wider range of plate girder geometries. Based on the results from both the experiments and the FE simulations, the obtained modes of failure were discussed and the shear design provisions of EN 1993-1-4 were assessed. Three different modes of failure were observed: a shear dominant failure, a bending dominant failure and a combined bending plus shear failure. It was found that the current Eurocode 3: Part 1.4 design equations are safely applicable to lean duplex stainless steel, but improved efficiency could be achieved through a revised design model and making distinction between plate girders with rigid and non-rigid end posts. Comparisons of the results obtained herein with others collected from the literature showed that this conclusion was also applicable to other stainless steel grades. Further research on this topic is underway.

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Fig. 1: Initial part of tensile and compressive stress-strain curves for 4 mm material (L= Longitudinal, T: Transverse).

Fig. 2: Initial part of tensile and compressive stress-strain curves for 6 mm material (L= Longitudinal, T: Transverse).

Fig. 3: Initial part of tensile and compressive stress-strain curves for 8 mm material (L= Longitudinal, T: Transverse).

Fig. 4: Initial part of tensile and compressive stress-strain curves for 10 mm material (L= Longitudinal, T= Transverse).

Fig. 5: Initial part of tensile and compressive stress-strain curves for 12 mm material (L= Longitudinal, T= Transverse).

Fig. 6: Initial part of tensile and compressive stress-strain curves for 15 mm material (L= Longitudinal, T= Transverse).

Fig. 7: Initial part of tensile and compressive stress-strain curves for 20 mm material (L= Longitudinal, T= Transverse).

Fig. 8: Geometry of the tested plate girders.

Fig. 9: Location of strain gauges.

Fig. 10: Schematic shear test setup and instrumentation.

Fig. 11: Load-mid-span vertical displacement curves for plate girders of aspect ratio 1.0.

Fig. 12: Load-mid-span vertical displacement curves for plate girders of aspect ratio 2.0.

Fig. 13: Load-lateral web displacement curves for plate girders of aspect ratio 1.0.

Fig. 14: Load-lateral web displacement curves for plate girders of aspect ratio 2.0.

Fig. 15: Typical shear dominant FE failure mode for plate girders with a web panel aspect ratio of 1.0.

Fig. 16: Typical shear dominant FE failure mode for plate girders with a web panel aspect ratio of 2.0.

Fig. 17: Experimental and numerical load-mid-span vertical displacement curve for the plate girder I-600×200×12×8-1.

Fig. 18: Experimental and numerical load-mid-span vertical displacement curve for the plate girder I-600×200×12×8-2.

Fig. 19: Evolution of principal strains at centre of web panel for plate girder I-600x200x12x8-1– comparison between experimental and numerical results.

Fig. 20: Evolution of principal strains at corner of web panel for plate girder I-600x200x12x8-1– comparison between experimental and numerical results.

Fig. 21: Inclination of principal plane to horizontal at centre of web panel for plate girder I-600×200×12×4-1– comparison between experimental and numerical results.

Fig. 22: Typical shear dominant failure mode showing shear buckling of the web for plate girder I-600×200×12×4-1.

Fig. 23: Moment-shear interaction diagram from EN 1993-1-4 [15] for plate girder I-600x200x12x4-1 exhibiting a shear dominant failure mode.

Fig. 24: Typical bending dominant failure mode showing local flange buckling for plate girder I-600x200x12x10-2.

Fig. 25: Moment-shear interaction diagram from EN 1993-1-4 [15] for plate girder I-600x200x12x10-2 exhibiting a bending dominant failure mode.

Fig. 26: Typical combined shear plus bending failure mode for plate girder I-600x200x12x8-1.

Fig. 27: Moment-shear interaction diagram from EN 1993-1-4 [15] for plate girder I-600×200×12×8-1 exhibiting a combined shear-bending failure mode.

Fig. 28: Moment-shear interaction and definition of a shear dominant failure as $V_{u,test}/M_{u,test} > V_{bw,Rd}/M_{f,Rd}$

Fig. 29: Comparison of experimental and numerical results for lean duplex stainless steel plate girders with rigid and non-rigid end posts and the EN 1993-1-4 shear design model.

Fig. 30: Comparison of shear buckling resistance for tested plate girders of various stainless steel grades.

Table 1: Measured material properties from tensile coupon tests, together with comparative transverse tension mill certificate values.

Plate thickness (mm)	Tensile coupon tests								Mill certificate values (transverse tension)		
	Orientation to rolling direction	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)	ϵ_f (%)	Modified R-O coefficients		$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)
							n	$n'_{0.2,1.0}$			
4.11	Longitudinal (L1)	191000	486	574	752	56	12.0	4.0	569	632	753
4.14	Transverse (T1)	208000	540	592	759	55	8.7	3.4	569	632	753
6.00	Longitudinal (L1)	207000	506	547	719	65	8.0	2.1	559	621	745
6.22	Transverse (T1)	205000	549	580	720	55	12.0	1.9	559	621	745
8.61	Longitudinal (L1)	203000	431	523	714	58	8.7	3.4	514	-	729
8.38	Transverse (T1)	209000	518	572	766	47	12.0	2.5	514	-	729
10.65	Longitudinal (L1)	204000	433	524	728	56	12.7	4.0	505	-	729
10.62	Transverse (T1)	205000	509	553	747	45	10.0	1.8	505	-	729
12.63	Longitudinal (L1)	218000	484	532	704	45	8.8	1.9	498	-	717
12.55	Transverse (T1)	204000	495	545	714	50	11.0	2.2	498	-	717
15.71	Longitudinal (L1)	205000	564	582	720	55	10.3	1.3	543	-	731
15.78	Transverse (T1)	218000	556	606	746	39	9.4	2.5	543	-	731
20.82	Longitudinal (L1)	214000	482	550	720	44	12.8	2.8	498	-	717
20.86	Transverse (T1)	219500	461	545	-	54	-	-	498	-	717

Table 2: Measured material properties from compressive coupon tests, together with comparative transverse tension mill certificate values.

Plate thickness (mm)	Compressive coupon tests						Mill certificate values (transverse tension)	
	Orientation to rolling direction	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	Modified R-O coefficients		$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)
					n	$n'_{0.2,1.0}$		
4.08	Longitudinal (L1)	201000	494	580	6.9	2.0	569	632
4.13	Transverse (T1)	213000	550	619	5.3	1.6	569	632
6.02	Longitudinal (L1)	195000	488	562	6.3	2.0	559	621
6.11	Transverse (T1)	212000	557	614	8.0	1.5	559	621
8.51	Longitudinal (L1)	-	-	-	-	-	514	-
8.6	Transverse (T1)	207000	492	569	5.5	1.9	514	-
10.65	Longitudinal (L1)	201000	461	539	6.0	1.9	505	-
10.68	Transverse (T1)	201000	500	570	6.1	1.8	505	-
12.57	Longitudinal (L1)	199000	447	521	5.3	1.7	498	-
12.54	Transverse (T1)	209000	488	569	6.0	1.7	498	-
15.78	Longitudinal (L1)	212000	488	573	6.2	1.6	543	-
15.79	Transverse (T1)	205000	509	591	6.0	1.8	543	-
20.92	Longitudinal (L1)	-	-	-	-	-	498	-
20.67	Transverse (T1)	202000	464	543	5.9	1.6	498	-

Table 3: Measured dimensions of plate girders and key test results.

Plate girder	L (mm)	a (mm)	e (mm)	h_w (mm)	b (mm)	t_f (mm)	t_w (mm)	t_s (mm)	b_s (mm)	a/h_w	$\bar{\lambda}_w$	V_{cr} (kN)	$V_{u, test}$ (kN)	$M_{u, test}$ (kNm)	c_{test} (mm)	Observed failure mode
I-600×200×12×4-1	1360	600	80.0	598.8	200.1	12.4	4.1	20.9	98.0	1.0	1.99	180	562	337	202	Shear
I-600×200×12×6-1	1360	600	80.0	599.9	199.8	12.3	6.2	20.6	96.8	1.0	1.32	626	888	888	193	Shear
I-600×200×12×8-1	1360	600	80.0	600.3	200.1	12.5	8.2	20.6	96.0	1.0	0.94	1484	1326	1326	-	Combined
I-600×200×12×10-1	1360	600	80.0	599.2	200.1	12.4	10.2	20.6	94.9	1.0	0.75	2877	1838	1838	-	Bending
I-600×200×12×4-2	2560	1200	80.0	600.0	200.1	12.6	4.1	20.8	98.0	2.0	2.41	122	396	475	285	Shear
I-600×200×12×6-2	2560	1200	80.0	600.9	200.0	12.3	6.0	20.8	97.0	2.0	1.61	425	682	818	315	Shear
I-600×200×12×8-2	2560	1200	80.0	600.0	199.8	12.3	8.4	20.6	95.7	2.0	1.14	1007	976	1172	-	Combined
I-600×200×12×10-2	2560	1200	80.0	600.1	200.4	12.6	10.6	20.9	94.9	2.0	0.91	1953	1162	1395	-	Bending
I-600×200×15×15-2	2560	1200	80.0	599.0	200.1	15.3	15.0	20.6	92.5	2.0	0.65	6817	1801	2162	-	Bending

Table 4: Comparison between experimental and numerical results.

Plate girder	$\omega_{D\&W}$ (mm)	a/h_w	$\bar{\lambda}_w$	$V_{u,test}$ (kN)	$V_{u,FE}$ (kN)	$V_{u,FE}/V_{u,test}$
I-600×200×12×4-1	0.25	1.00	1.99	562	596	1.06
I-600×200×12×6-1	0.16	1.00	1.32	888	893	1.01
I-600×200×12×8-1	0.11	1.00	0.94	1326	1336	1.01
I-600×200×12×10-1	0.088	1.00	0.75	1838	1753	0.95
I-600×200×12×4-2	0.25	2.00	2.41	396	406	1.03
I-600×200×12×6-2	0.16	2.00	1.61	682	700	1.03
I-600×200×12×8-2	0.11	2.00	1.14	976	954	0.98
I-600×200×12×10-2	0.088	2.00	0.91	1162	1152	0.99
I-600×200×15×15-2	0.067	2.00	0.65	1801	1747	0.97
Mean						1.00
COV						0.03

Table 5: Comparisons of experimental results with EN 1993-1-4 predictions.

Plate girder	a/h_w	$\bar{\lambda}_w$	$V_{u,test}$ (kN)	$V_{EN\ 1993-1-4}$ (kN)	$V_{u,test}/V_{EN\ 1993-1-4}$
I-600×200×12×4-1	1.00	1.99	562	386	1.46
I-600×200×12×6-1	1.00	1.32	888	693	1.28
I-600×200×12×8-1	1.00	0.94	1326	999	1.33
I-600×200×12×10-1	1.00	0.75	1838	1274	1.44
I-600×200×12×4-2	2.00	2.41	396	297	1.33
I-600×200×12×6-2	2.00	1.61	682	546	1.25
I-600×200×12×8-2	2.00	1.14	976	652	1.50
I-600×200×12×10-2	2.00	0.91	1162	699	1.66
I-600×200×15×15-2	2.00	0.65	1801	1489	1.21
Mean					1.38
COV					0.07

Table 6: Comparisons of parametric FE results on plate girders with rigid end posts with the EN 1993-1-4 predictions.

Plate girder	a/h_w	$\bar{\lambda}_w$	$V_{u,FE}$ (kN)	$V_{EN\ 1993-1-4}$ (kN)	$V_{u,FE}/V_{EN1993-1-4}$
I-700×200×60×30-0.86	0.86	0.26	8050	6655	1.21
I-700×200×60×25-0.86	0.86	0.31	7050	5944	1.19
I-700×200×60×20-0.86	0.86	0.38	5900	4963	1.19
I-600×200×15×12-1	1.00	0.63	2056	1917	1.07
I-600×200×15×10-1	1.00	0.75	1708	1473	1.16
I-600×200×12×3-1	1.00	2.65	443	270	1.64
I-600×200×12×2-1	1.00	3.98	303	171	1.77
I-600×200×15×6-2	2.00	1.61	731	583	1.25
I-600×200×15×4-2	2.00	2.41	428	323	1.33
I-600×200×15×3-2	2.00	3.22	307	222	1.38
I-600×200×15×2-2	2.00	4.83	204	136	1.50
I-600×200×12×3-2	2.00	3.22	281	202	1.39
I-600×200×12×2-2	2.00	4.83	179	123	1.46
Mean					1.35
COV					0.15

Table 7: Comparisons of parametric FE results on plate girders with non-rigid end posts with the EN 1993-1-4 predictions.

Plate girder	a/h_w	$\bar{\lambda}_w$	$V_{u,FE}$ (kN)	$V_{EN\ 1993-1-4}$ (kN)	$V_{u,FE}/V_{EN1993-1-4}$
I-600×200×15×12-1	1.00	0.63	2074	1917	1.08
I-600×200×15×10-1	1.00	0.75	1734	1473	1.18
I-600×200×12×10-1	1.00	0.75	1823	1344	1.36
I-600×200×12×8-1	1.00	0.94	1323	999	1.32
I-600×200×12×6-1	1.00	1.32	830	693	1.20
I-600×200×12×4-1	1.00	1.99	519	387	1.34
I-600×200×12×3-1	1.00	2.65	375	270	1.39
I-600×200×12×2-1	1.00	3.98	267	171	1.56
I-600×200×15×6-2	2.00	1.61	714	583	1.22
I-600×200×15×4-2	2.00	2.41	411	323	1.27
I-600×200×15×3-2	2.00	3.22	290	222	1.31
I-600×200×15×2-2	2.00	4.83	194	129	1.51
I-600×200×15×15-2	2.00	0.65	1747	1489	1.17
I-600×200×12×10-2	2.00	0.91	1151	883	1.30
I-600×200×12×8-2	2.00	1.14	950	725	1.31
I-600×200×12×6-2	2.00	1.61	688	559	1.23
I-600×200×12×4-2	2.00	2.41	390	298	1.31
I-600×200×12×3-2	2.00	3.22	268	202	1.33
I-600×200×12×2-2	2.00	4.83	170	123	1.39
Mean					1.30
COV					0.09

Table 8: Comparison between FE results for rigid and non-rigid end post plate girders.

Plate girder	a/h_w	$\bar{\lambda}_w$	$V_{u,FE,rigid}$ (kN)	$V_{u,FE,non-rigid}$ (kN)	$V_{u,FE,rigid}/V_{u,FE,non-rigid}$
I-600×200×12×2-1	1.00	3.98	303	267	1.14
I-600×200×12×3-1	1.00	2.65	443	375	1.18
I-600×200×12×4-1	1.00	1.99	596	519	1.15
I-600×200×12×6-1	1.00	1.32	893	830	1.08
I-600×200×12×8-1	1.00	0.94	1336	1323	1.01
I-600×200×12×2-2	2.00	4.83	179	170	1.05
I-600×200×12×3-2	2.00	3.22	281	268	1.05
I-600×200×12×4-2	2.00	2.41	406	390	1.04
I-600×200×12×6-2	2.00	1.61	700	688	1.02
I-600×200×12×8-2	2.00	1.14	954	950	1.00
I-600×200×12×10-2	2.00	0.91	1152	1151	1.00
I-600×200×15×15-2	2.00	0.65	1747	1747	1.00
I-600×200×15×12-1	1.00	0.63	2056	2074	0.99
I-600×200×15×10-1	1.00	0.75	1708	1734	0.99
I-600×200×15×6-2	2.00	1.61	731	714	1.02
I-600×200×15×4-2	2.00	2.41	428	411	1.04
I-600×200×15×3-2	2.00	3.22	307	290	1.06
I-600×200×15×2-2	2.00	4.83	204	194	1.05
Mean					1.05
COV					0.05