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Shear response of lean duplex stainless steel plate girders

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Abstract. Carbon steel plate girders have been used on a large scale in the building industry. Nowadays, Lean Duplex Stainless Steel (LDSS) plate girders are gaining popularity as they possess greater strength and are more impervious to corrosion than those that are constructed from carbon steel. Regardless of their popularity, there is very limited information with regards to their shear behavior. In this paper, the non-linear finite element analysis was employed to investigate the shear behavior of LDSS plate girders. Parameters considered were the web thickness, the flange width, and the girders aspect ratio. The analysis revealed that although the shear behavior of the LDSS girders was no different from that of carbon steel plate girders, it had obviously been affected by the non-linearity of the material. Furthermore, the selected parameters were found to pronounce effect on the shear capacity of the LDSS girders. That is, the shear capacity increased considerably with web thickness, and increased slightly with flange width. However, it was reduced as the aspect ratio increased. Comparisons between the finite element analysis failure loads and those predicted by the current European Code of Practice revealed that the latter underestimated the shear strength of the LDSS plate girders.

Keywords: lean duplex stainless steel (LDSS); shear response; ultimate shear capacity; finite element analysis; Welded I-sections

1. Introduction

The modern duplex stainless steel, which is produced from cast alloys, has been a common engineering material since its appearance in the early 1980s. It is popular because it possesses an appealing combination of properties, such as durability and a tremendous resistance to corrosion cracking under chloride stress. Since then, various grades of duplex have been developed and there has been an increase in its production. However, duplex stainless steels are recognized as a possible substitute for various other types of stainless steel and nickel based alloys. This rapid growth in production has led to an increase in researches into the subject (Gunn 1997).

Generally, the construction industry focuses primarily on the austenitic grades. EN 1.4301/1.4307 and EN 1.4401/1.4404, which contain roughly 8-11% of nickel, are the most

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frequently used grades of austenitic stainless steel. As nickel balances the austenitic microstructure, it adds to the related positive features such as formability, weldability, toughness, and high temperature attributes. However, much of the cost of austenitic stainless steel is mainly due to the presence of nickel. Therefore, due to the recent high prices of nickel, there has been a greater demand for lean duplexes with low nickel content (around 1.5%), such as EN 1.4162 grade (Gardner 2005).

The design of stainless steel parts is more complex than that of carbon steels because of the differences between their mechanical behaviors. The stress-strain curves of stainless steels have a gradual yield with fairly low proportional limits. Osgood and Ramberg (1943) proposed a simple formula for describing the stress-strain curve by using three parameters: namely, Young's modulus and two secant yield strengths.

Unosson and Olsson (2003) carried out experimental and theoretical study to investigate the resistance of welded I-girders made of stainless subjected to concentrated force and shear. They concluded that ENV 1993-1-5:1997 is in better agreement with the results from this study than ENV 1993-1-1:1992, and can be used for predicting the resistance to concentrated loads or shear.

Rasmussen (2003) developed an expression for the stress-strain curves for stainless steel alloys, which can be applied over the full range of strains. The expression can be used for the design and numerical modeling of stainless steel parts and elements which undergo stresses beyond the 0.2% proof stress in their final limit state. In this stress range, the current stress-strain curves, which are based on the Ramberg-Osgood expression, become extremely inaccurate mainly because these curve extrapolations are meant for stresses lower than the 0.2% proof stress. The extrapolations become markedly inaccurate for alloys with obvious strain hardening. Using his experimental data and data from previous researchers, Rasmussen was able to ascertain the full range of the stress-strain curve, which is employed in the numerical part of this paper.

In designing lean steel plated structures, one of the most vital load issues to be taken into consideration is resistance to shear. However, very little research has been carried out with regard to this issue in stainless steel structures and this material has only recently made its appearance in the construction industry. An experimental and numerical study was conducted in the Laboratory of Structural Technology of the Department of Construction Engineering in UPC by Real *et al*. (2007) to investigate the effect of shear load on stainless steel plated girders at approximate service conditions and what happens to the girders until failure occurs. The results of the experiment were compared to those obtained from the application of ENV 1993-1-4 and those provided by the numerical model.

In 2007, experiments were conducted at UPC by Estrada *et al*. (2007 a, b) as part of a wide ranging study on the shear behavior of stainless steel girders aimed at understanding how stainless steel plate girders react to shear loads. Theofanous and Gardner (2010) focused their research on lean duplex stainless steel hollow sections by performing an FE analysis. Since 2010, the study of stainless steel plate girders has become more widespread because of the increasing importance of stainless steel within the building industry. Hassanein (2010) performed a study on the features of the shear failure mechanism in stainless steel plate girders.

Hassanein (2011) recently introduced a theoretical set of models to examine the shear failure of the lean duplex stainless steel Grade EN 1.4162 plate girders. Longshithung Patton and Singh (2012) conducted tests on the use of LDSS (lean duplex stainless steel) in hollow columns of varying cross-sectional shapes under pure axial compression.

Saliba and Gardner (2013) conducted an experimental and numerical study at the Imperial College, London to examine the structural behavior of lean duplex stainless steel welded I-sections as well as the shear response of lean duplex stainless steel plate girders. The modes of failure that were obtained from the outcomes of both the experiments and the FE simulations were discussed and the shear design provisions of EN 1993-1-4 were evaluated. However, the FE validation in this paper has been compared with the experimental data obtained from the study by Saliba and Gardner (2013).

Saliba *et al*. (2014) reviewed the previous researched conducted to investigate the behavior and design of stainless steel plate girders loaded in shear. They 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*. (2007 a, b) and EN 1993-1-5.

The finite element method (FEM) was employed in this study to examine the shear behavior of LDSS girders. The following factors were taken into consideration: the web thickness, the flange width, and the girders aspect ratio. Therefore, the current study presents a set of models to explore the shear response and strength of lean duplex stainless steel grade EN 1.4162 rigid end stiffeners. As such, the impact of various factors such as flange width to web depth ratio (b_f/h_w), flange to web thickness ratio (t_f/t_w), as well as web height to web thickness ratio (h_w/t_w) have been taken into account.

2. Finite element model

2.1 General

In order to examine the shear response of the nonlinear lean duplex stainless steel (LDSS) plate girders, a numerical analysis was carried out on the actual plate girders. The current model using the finite element program was based on the experimental program that presented by Saliba and Gardner (2013) for lean duplex stainless steel plate girders with square web panels. In total, 24 plate girder models were analyzed throughout the numerical investigation by employing the LUSAS V14.3. The following features were investigated: web thickness, flange width, and aspect ratio. The performances of these variables were also calculated according to the following parameters:

1. Flange to web thickness ratio (t_f/t_w); (2, 2.4, 3 and 4),
2. Flange width to web depth ratio (b_f/h_w); (0.25, 0.33 and 0.41),
3. Web plate slenderness (h_w/t_w); (100, 120, 150, and 200).

For all the finite element models in this study, the web depth of the plate girders was fixed at 600 mm, while the span of the plate girders for the aspect ratio=1 and aspect ratio=2 were 1200 and 2400, respectively. The particulars of the parametric studies with both aspect ratios are shown in Table 1. The numerical investigation was divided into three groups according to the flange width (b_f). The first group had a flange width of 150 mm, the second group had a flange width of 200 mm, and the third group had a flange width of 250 mm. The web thickness for each group varied between 3 to 6 mm. The same details above apply to both aspect ratios 1 and 2.

The finite element analysis program comprised three groups. The lean duplex stainless steel plate girders (LDSS) were labeled in such a way that the aspect ratio number could be recognized from the label. The group was represented by the letter G and this was followed by the group number and by numbers in parentheses, which gave the thickness of the flange and the web in millimeters separated by a dash. For example, the label ‘‘LDSS1G1 (12−4)’’ would describe a

Table 1 Finite element program

Group	b_f (mm)	b_f/h_w^*	t_w (mm)	h_w/t_w	t_f (mm)	t_f/t_w
LDSSG1	150	0.25	3, 4,5, 6	200, 150, 120, 100	12	4, 3,2.4, 2
LDSSG2	200	0.33	3, 4,5, 6	200, 150, 120, 100	12	4, 3,2.4, 2
LDSSG3	250	0.41	3, 4,5, 6	200, 150, 120, 100	12	4, 3,2.4, 2

*h_w=600 mm

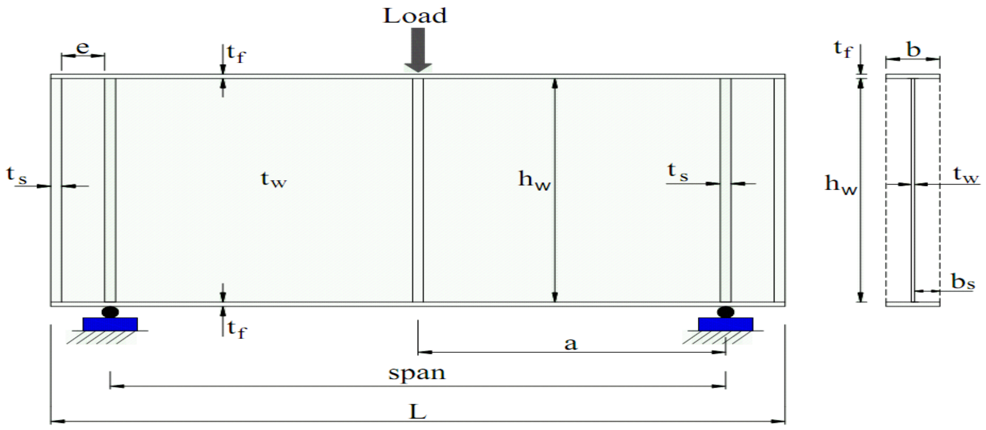


Fig. 1 Geometry of the analyzed plate girders

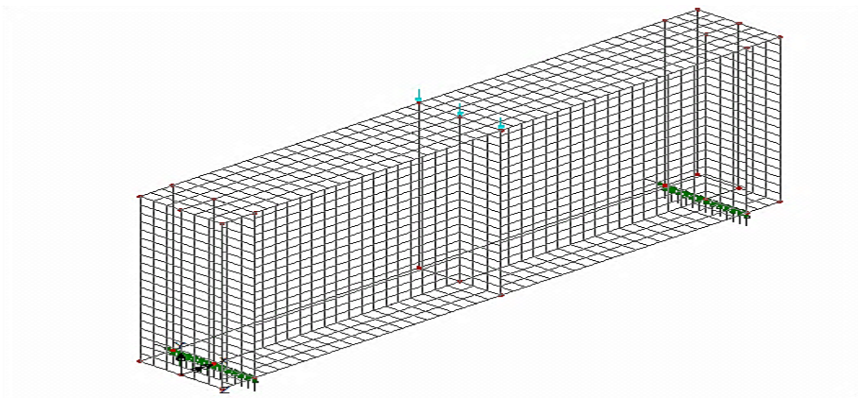


Fig. 2 Typical finite element modeling used in the analyses

girder with an aspect ratio=1.0 and belonging to group 1, with a flange and web thickness of 12 and 4 mm, respectively. Table 1 lists the details of the groups and the aspect ratios. Throughout the research the thickness of the transverse stiffeners was taken as 20 mm (see Fig. 1).

2.2 Finite element type and mesh

LUSAS V14.3 software was used for the three dimensional finite-element models of the whole plate girder. The finite-element mesh was made up of shell elements in the web, stiffener plates and flanges. In this study, the normal thin shell element, QSL8, was accepted for the analyses of the plate girders. The lean duplex stainless steel (LDSS) can be represented by thin shells as the behavior of the former can be accurately illustrated in terms of its distorted center. This suggests, therefore, that out of the three constituent dimensions, two of them, i.e. the length and width, must be larger than the third dimension, which is the thickness. Since the assumption is that the element is thin, the thickness stresses are deemed to be zero throughout. This type of element, with 8 nodes and 5 degrees of freedom per node, is particularly suitable for reproducing issues concerning large displacements and small strains.

A 30 mm size mesh was used in this study. The size of mesh was selected after convergence studies. For models with aspect ratios 1 and 2, the total elements were 2072 and 3352, respectively. (see Fig. 2).

2.3 Boundary conditions and load applications

The boundary conditions were thoroughly integrated into the FE models in order to reproduce the experimental setup. The plate girder models that were being considered in this study were put through concentrated loads at their mid-spans. The corresponding load was applied in a number of load steps. The equilibrium iteration was done at each load step and the equilibrium patch was traced in the load-displacement space. The loads were applied as static uniform loads at the loaded point using displacement control. To handle the large displacement, the nonlinear geometry parameter (NLGEOM) was added in.

At the supports, the vertical and lateral displacements were restrained, while at the mid-span the longitudinal displacements and rotation about the horizontal axis were controlled (see Fig. 2).

2.4 Lean duplex stainless steel material modeling

Carbon steel and Lean Duplex Stainless Steel (LDSS) differ mainly in their stress-strain relationship. Unlike carbon steel, which displays a linear elastic behavior with a distinct yield point, lean duplex stainless steel (LDSS) responds in a rounded and anisotropic strain-stress way without a clearly marked yield point. Rasmussen (2003) came up with the stress-strain equations of stainless steel according to the Ramberg-Osgood equation as in Eq. (1). These equations were used in this study to create the stress-strain curve of the Grade EN 1.4162 lean duplex stainless steel (LDSS) material, as shown in Fig. 3.

$$\varepsilon = \begin{cases} \frac{\sigma}{E_0} + 0.002\left(\frac{\sigma}{\sigma_{0.2}}\right)^n & for\ \sigma \leq \sigma_{0.2} \\ \\ \frac{\sigma-\sigma_{0.2}}{E_{0.2}} + \varepsilon_u\left(\frac{\sigma-\sigma_{0.2}}{\sigma_u-\sigma_{0.2}}\right)^m + \varepsilon_{0.2} & for\ \sigma \geq \sigma_{0.2} \end{cases} \tag{1}$$

where ε=engineering strain, σ=engineering stress, $\sigma_{0.2}$=tensile 0.2% proof stress, n=nonlinearity index σ_u=ultimate tensile strength, ε_u=ultimate strain, $\varepsilon_{0.2}$=0.2% total strain, E_0 is the initial modulus of elasticity (e.g., 206 GPa) and $E_{0.2}$ is the tangent modulus of the stress-strain curve at the 0.2% proof stress and given as

$$E_{0.2} = \frac{E_0}{1+0.002\ n/e} \tag{2}$$

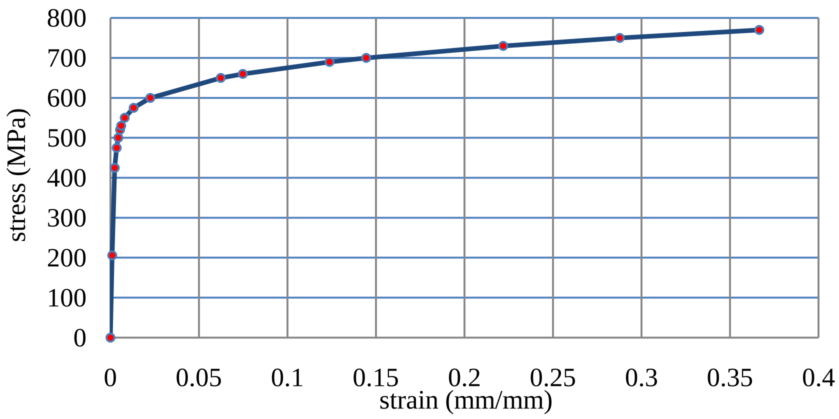


Fig. 3 Stress-strain curve of the lean duplex stainless steel material grade EN1.4162. (Eq. (1))

Where, e is the non-dimensional proof stress given as $e=\sigma_{0.2}/E_0$. The other parameters can be found in Rasmussen (2003).

The von Mises yielding surface with isotropic hardening was used to describe the plastic range. When there is absolute plasticity, the yielding stress will not change with the plastic strain. However, with isotropic hardening, the yielding surface will increase consistently in all directions when there is an increase (or decrease) in the yield stress once there is plastic strain. Structural steel is an elastic, absolutely plastic material in terms of tension and compression having a Young's Modulus of 206 KN/mm^2 and Poisson's ratio of 0.3, as well as $\sigma_{0.2}$=500 N/mm^2, $\sigma_{1.0}$=580 N/mm^2 and n=10. It has been used in FE convergence studies and the parametric studies have been according to the experimental data provided by Saliba and Gardner (2013).

2.5 Validation of models

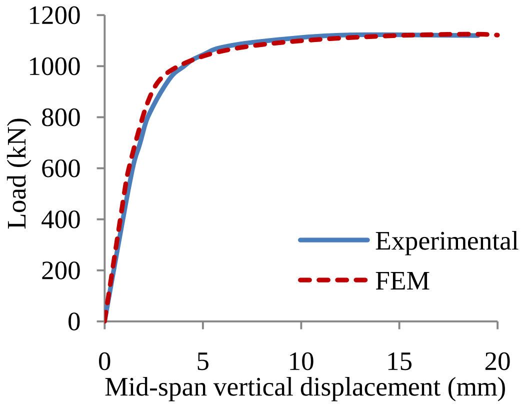
The simulation process had to be validated as to its accuracy. The plate girders were also replicated and validated by the experimental data in Saliba and Gardner (2013). The girder that was verified in Fig. 4(a) was originally as I-600×200×12×4-1 in Saliba and Gardner (2013). The girder spanned 1360 mm with an aspect ratio of 1, and the thickness of the flange, the web, and the stiffeners were 12, 4, and 20 mm, respectively (see Table 2). Table 3 shows the final load that was obtained in numerical terms compared to the tested load provided in Saliba and Gardner (2013). Furthermore, another girder, called I-600×200×12×4-2, was also numerically created (see Fig. 4(b)). This girder spanned 2560 mm with an aspect ratio of 2, and the thickness of the flange, the web, and the stiffeners were 12, 4, and 20 mm, respectively (see Table 2). The experimental results and the current numerical modeling results were very much in agreement. The load-mid-span

Table 2 Dimensions and material properties of the analyzed girders

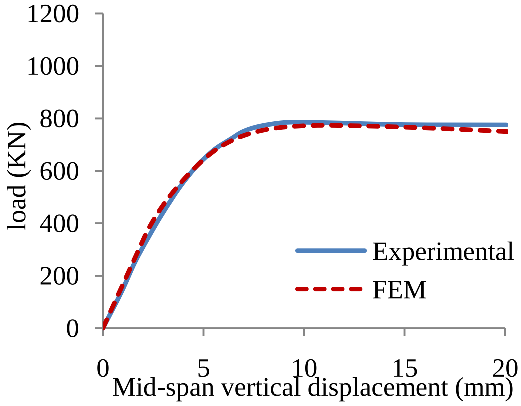
Specimen	L (mm)	a (mm)	e (mm)	h_w (mm)	b (mm)	t_f (mm)	t_w (mm)	t_s (mm)	b_s (mm)	a/h_w	E_0 (N/mm^2)	$\sigma_{0.2}$ (N/mm^2)
I-600×200×12×4-1	1360	600	80.0	600	200	12	4	20	98	1	206,000	500
I-600×200×12×4-2	2560	1200	80.0	600	200	12	4	20	98	2	206,000	500

Table 3 Finite element results of plate girders tested by Saliba and Gardner (2013)

Specimen	F_{Test} (KN)	F_{FE} (KN)	F_{FE}/F_{Test}
I-600×200×12×4-1	562	562.5	1.004
I-600×200×12×4-2	396	386.6	0.97
		Ave	0.99
		SD	0.024



(a) Model with aspect ratio=1



(b) Model with aspect ratio=2

Fig. 4 Validation of finite element models with aspect ratio 1 and 2

vertical displacement curves for both plate girders with aspect ratios 1 and 2 respectively can be seen in Fig. 4.

3. Analysis of results

3.1 Introduction

In this section, the outcomes of the numerical analyses are assessed in order to study the impact of material nonlinearity on the strength and behavior of lean duplex stainless steel plate girders under shear. The finite element strengths (F_{FE}) and ultimate shear loads (V_{FE}) of the lean duplex stainless steel plate girders Grade 1.4162 are presented in Tables 4-5. The responses and modes of failure in shear have been monitored and evaluated in detail. The analysis was specifically focused on the impact of the aspect ratio, variance in the web thickness and variance of flange width since these are the main targets of this study.

3.2 Failure mechanism

The behavior of the lean duplex stainless steel (LDSS) girders were demonstrated to be the same as those made of carbon steel, but obviously subject to the non-linearity of the material. The general behavior of the girder can be identified from the load-deflection curves, and according to the general buckling and tension field theory, the buckling will most likely begin at the center of

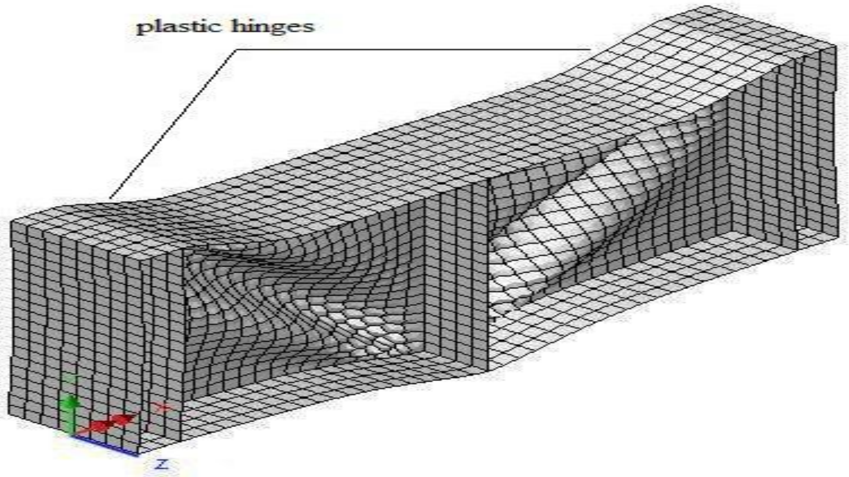


Fig. 5 FE shear failure mechanism for the analyzed plate girder

Table 4 Results of the finite element program for aspect ratio=1.0

Specimen	b_f/h_w^*	h_w/t_w	t_f/t_w	F_{FE} (KN)	V_{FE} (KN)
LDSS1G1 (12 − 3)	0.25	200	4	779.6	389.8
LDSS1G1 (12 − 4)	0.25	150	3	1027	513.5
LDSS1G1 (12 − 5)	0.25	120	2.4	1320.8	660.4
LDSS1G1 (12 − 6)	0.25	100	2	1660.45	830.225
LDSS1G2 (12 − 3)	0.33	200	4	805	402.5
LDSS1G2 (12 − 4)	0.33	150	3	1047.7	523.85
LDSS1G2 (12 − 5)	0.33	120	2.4	1360.9	680.45
LDSS1G2 (12 − 6)	0.33	100	2	1696.35	848.175
LDSS1G3 (12 − 3)	0.41	200	4	833	416.5
LDSS1G3 (12 − 4)	0.41	150	3	1133.6	566.8
LDSS1G3 (12 − 5)	0.41	120	2.4	1398	699
LDSS1G3 (12 − 6)	0.41	100	2	1719	859.5

*h_w=600 mm, a/h_w=1

the panel. Furthermore, once the buckling happens, the stresses will be redistributed into the tension band, which must be reinforced by securing these forces in the flanges and stiffeners. After buckling occurs, the plate cannot carry further compressive stresses and a new load carrying mechanism develops, whereby any additional shear loading is supported by an inclined tensile stress field. As the applied loading increases, the tensile membrane stress grows until it reaches the yield stress of the material. When the web has yielded, final collapse will occur when plastic hinges are formed in the flanges that permit a shear sway failure mechanism (see Fig. 5). All the girder samples failed in shear. It is a well-known fact that web plates may display a great deal of post-critical strength reserve that must be given due consideration in the designing of steel plate girders. This occurrence was first investigated by Wilson (1886) and various other researchers, including Rockey and Skaloud (1972), later expounded it.

Table 5 Results of the finite element program for aspect ratio=2.0

Specimen	b_f/h_w^*	h_w/t_w	t_f/t_w	F_{FE} (KN)	V_{FE} (KN)
LDSS2G1 (12 − 3)	0.25	200	4	517	258.5
LDSS2G1 (12 − 4)	0.25	150	3	727.4	363.7
LDSS2G1 (12 − 5)	0.25	120	2.4	1044	522
LDSS2G1 (12 − 6)	0.25	100	2	1309	654.5
LDSS2G2 (12 − 3)	0.33	200	4	536	268
LDSS2G2 (12 − 4)	0.33	150	3	773	386.5
LDSS2G2 (12 − 5)	0.33	120	2.4	1075.7	537.85
LDSS2G2 (12 − 6)	0.33	100	2	1408.77	704.4
LDSS2G3 (12 − 3)	0.41	200	4	540	270
LDSS2G3 (12 − 4)	0.41	150	3	785	392.5
LDSS2G3 (12 − 5)	0.41	120	2.4	1083.7	541.85
LDSS2G3 (12 − 6)	0.41	100	2	1430.8	715.4

*h_w=600 mm, a/h_w=2.0

3.3 Load-displacement relationship of the plate girders

In order to investigate the structural behavior of lean duplex stainless steel (LDSS) plate girders, parametric studies must be carried out on the relationship between the load and the mid-span vertical displacement, together with the aspect ratios 1 and 2 (see Figs. 6(a)-(b)). From these curves, it is possible to detect the main changes in the behavior of the girder from the variations in the parameters such as web thickness, flange width-to-depth ratio (b_f/h_w), web slenderness (h_w/t_w), flange-to-web thickness ratio (t_f/t_w) as well as the aspect ratio. The results indicate that the flange width-to-web depth ratio (b_f/h_w) has an effect on the failure modes of the duplex stainless steel plate girders. When the flange width-to-web depth ratio (b_f/h_w) is 0.25 and the flange-to-web thickness ratio (t_f/t_w) for both aspect ratios is more than 2, then the failure mode is always in shear. However, when the flange width-to-web depth ratio (b_f/h_w) is 0.30 or 0.41, regardless of the slenderness of the web plate (h_w/t_w), and when the flange-to-web thickness ratio (t_f/t_w) for both aspect ratios is more than 2, the failure mode is always in shear. It should be noted that the models in Fig. 6 have a flange width of (b_f=200 mm).

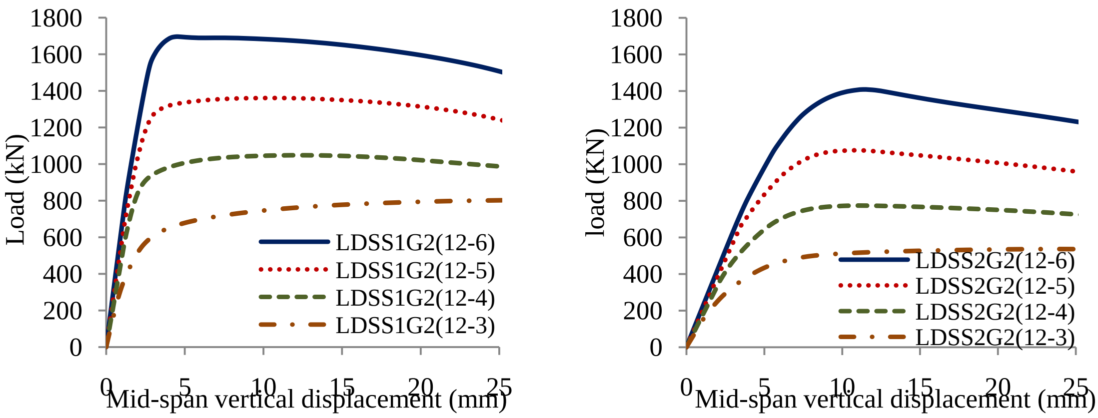
3.4 Effect of chosen parameters

Once the obtained numerical results were in good general agreement with the experimental results, a sequence of chosen parameters was introduced to evaluate the general behavior of the lean duplex stainless steel plate girders.

3.4.1 Web thickness (t_w)

Fig. 7 shows the effect of web thickness on the ultimate shear loads of lean duplex stainless steel (LDSS) plate girders. It can be observed that any increase in the thickness of the web led to an increase in the ultimate load shear as well.

However, since the flange thickness for all the FE models was fixed at 12 mm, the structural behavior of the lean duplex stainless (LDSS) plate girders was influenced by the flange-to-web



(a) Aspect ratio=1 (b) Aspect ratio=2

Fig. 6 Load-mid-span vertical displacement curves of plate girders (b_f=200 mm)

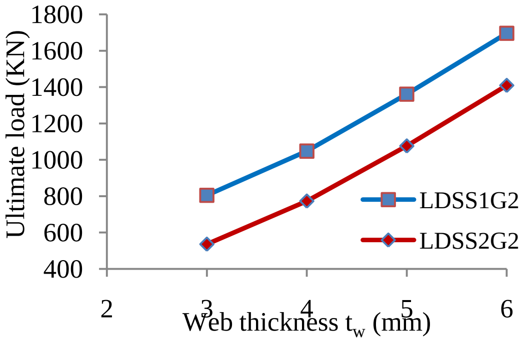


Fig. 7 Ultimate load-web thickness (t_w) for aspect ratio1and 2 (b_f=200 mm)

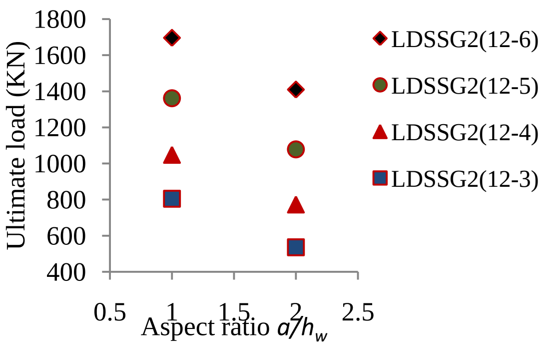


Fig. 8 Ultimate load-aspect ratio (a/h_w) for group 2 (b_f=200 mm)

thickness ratio (t_f/t_w). Reducing this ratio would enable the girder to attain greater strength and ductility. As it can be concluded from Table 4-5 that when the web-depth-to-web thickness ratio (h_w/t_w) decreases, there is an increase in the resistance of the plate girders.

3.4.2 Flange width (b_f)

The rotated stress field method, based on Hoglund's theory (1997) was proposed for the EN 1993-1-5. The design terms included the flange contribution ($V_{bf,Rd}$) in the resistant mechanism that allowed a secured tension field to be created so as to raise the shear capacity.

The effect of flange contribution has been examined in the current study and it was observed that the flange contribution can affect and raise the ultimate shear loads. Since this study is only focused on changes to the flange width, therefore the flange contribution has only a slight effect on variations in the flange width (150, 200 and 250 mm). The results also illustrate that with the web depth fixed at 600 mm, an increase in the flange width-to-web depth (b_f/h_w) leads to an increase in the resistance of the plate girders, as seen in Tables 4-5.

3.4.3 Aspect ratio (a/h_w)

It is crucial that the effect of the aspect ratio of the plate girders has been investigated as the aspect ratio of the web panel has a great effect on the shear failure of lean duplex stainless steel (LDSS) plate girders. This work has been carried out and both aspect ratios 1 and 2 have been discussed.

One interesting observation is that those panels with equal values of web slenderness (h_w/t_w) but different aspect ratios displayed different strengths according to the web contribution. In fact, the higher aspect ratio values contributed less of the web panel (χ_w) to the whole resistance mechanism (see Fig. 8).

It was noted that for the slender lean duplex stainless steel (LDSS) plate girders (LDSS2G1, 2, 3 (12-3)), shear buckling almost occurred along the linear path. Hence, geometrical nonlinearity was the cause of the failure because the plastic shear resistance for that plate girder was less than the maximum elastic applied load (623 KN) at which the web is still under pure shear for the web thickness of 3 mm, which is calculated according to the following equation:

$$\tau = \frac{F/2}{h_w t_w} \tag{3}$$

The von Mises stress ($\sigma_{vm} = \tau\sqrt{3}$) may be taken to be equal to $\sigma_{0.01}$=300 MPa. In that girder, the impact of the material non-linearity occurred later during the creation of the tension field in the post-critical range.

In the somewhat sturdy and thick girders, buckling happened in the nonlinear range of the duplex stainless steel material. The material nonlinearity effect occurred because the final loads that were applied to these girders were greater than the maximum elastic load (623 KN) that was applied.

3.5 Comparison with design code (EN 1993-1-4)

This comparison involved the EN 1.4162 lean duplex stainless steel (LDSS) girders depicted by shear failure. As shown in Table 6, the EN 1993-1-4 was used to make a comparison between the finite element ultimate shear force and the factored design shear resistance.

The design shear resistance, $V_{c,Rd}$, was assumed to be the lesser of the shear buckling resistance, $V_{b,Rd}$, , based on Clause 5.2(1) of EN 1993-1-5 modified by (3) and (4) and the plastic resistance, $V_{pl,Rd}$, based on Clause 6.2.6.of EN 1993-1-1. Nevertheless, the rotated stress field method, which is based on Hoglund's theory (1997), was suggested in EN 1993-1-4. The design terms included the flange contribution ($V_{bf,Rd}$) in the resistant mechanism that allowed the creation of a secured tension field that raised the shear capacity.

Table 6 Comparison between ultimate shear (V_{FE}) and design shear resistance according to EN 1993-1-4

Specimen	a/h_w=1			a/h_w=2		
	V_{FE} (KN)	$V_{b,Rd}$ (KN)	$V_{b,Rd}/V_{FE}$	V_{FE} (KN)	$V_{b,Rd}$ (KN)	$V_{b,Rd}/V_{FE}$
LDSSG1(12 − 3)	389.8	255	0.65	258.5	194	0.75
LDSSG1(12 − 4)	513.5	381	0.74	363.7	299	0.82
LDSSG1(12 − 5)	660.4	520	0.78	522	410	0.78
LDSSG1(12 − 6)	830.22	670	0.80	654.5	534	0.81
LDSSG2(12 − 3)	402.5	274	0.68	268	204	0.76
LDSSG2(12 − 4)	523.85	405	0.77	386.5	313	0.80
LDSSG2(12 − 5)	680.45	551	0.80	537.85	432	0.80
LDSSG2(12 − 6)	848.17	708	0.83	704.4	559	0.79
LDSSG3(12 − 3)	416.5	288	0.69	270	213	0.78
LDSSG3(12 − 4)	566.8	423	0.74	392.5	325	0.82
LDSSG3(12 − 5)	699	574	0.82	541.85	450	0.83
LDSSG3(12 − 6)	859.5	737	0.85	715.4	583	0.81
		Ave	0.77			0.80
		SD	0.06			0.02

Based on the EN 1993-1-5, the design resistance for shear ($V_{b,Rd}$) for unstiffened or stiffened webs should be calculated as

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t_w}{\sqrt{3}\gamma_{m1}} \tag{4}$$

The web and flange contributions are given below in Eqs. (5)-(6), respectively

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3}\gamma_{m1}} \tag{5}$$

$$V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c\gamma_{m1}}\left(1 - (\frac{M_{Ed}}{M_{f,Rd}})^2\right) \tag{6}$$

Where, χ_w is the contribution from the web to shear buckling resistance and can be obtained from Table 5.1 or Figure 5.2 of EN 1993-1-5, which is given as $\chi_w = 1.37/(0.7 + \bar{\lambda}_w)$. The modified slenderness, $\bar{\lambda}_w$, of EN 1993-1-5 in Table 5.1 may be calculated as

$$\bar{\lambda}_w = \frac{h_w}{37.4\, t\, \epsilon \sqrt{k_\tau}} \tag{7}$$

Where, k_τ is the minimum shear buckling coefficient of the web panel and can be obtained from clause A.3 of EN 1993-1-5, which is given as $k_\tau = 5.34 + 4(h_w/a)^2$ and γ_{m1} is taken as (1.1) according to EN 1993-1-5.

The contribution from the flanges is determined by the moment (M_{Ed}), which is regarded in this case as the largest moment within the end panel, i.e. the distance from the support. ($M_{f,Rd}$) is the moment of resistance of the cross-section comprising only of the area of the effective flanges ($M_{f,Rd}$=$A_f d f_{yf}$), d is the distance between the centroids of the flanges, and c is the supposed distance between the plastic hinge and the end stiffener, which for stainless steel plate girders can be ascertained from EN 1994-1-4 as

$$c = \left(0.17 + \frac{3.5\ .b_f t_f^2 f_{yf}}{t_w d^2 f_{yf}}\right).a. \tag{8}$$

Annex *A*, Table A1 gives a sample of the key factors that are employed in the calculations. As can be seen, there is good agreement between the finite element strengths with the design specifications of EN 1993-1-4, and the shear strength has been overestimated a bit based on the EN 1993-1-4 of the LDSS plate girders.

4. Conclusions

The following conclusions are drawn from the numerical studies conducted herein:

• By comparing the numerical analysis that was carried out in this study and the experimental results that were stated in the literature, it can be shown that the numerical results that were obtained were in good general agreement with the experimental results.

• The web thickness was found to have a noticeable effect on the shear capacity of lean duplex stainless steel (LDSS) plate girders. When compared with sample (I-600×200×12×4-1), which had a web thickness of 4 mm, the ultimate shear capacity of the samples increased by 23% and 21% when the web thickness increased to 5 mm and 6 mm, respectively.

• The effect of the ratio of the flange thickness to the web thickness, t_f/t_w on the shear behavior of lean duplex stainless (LDSS) plate girders was examined. The results indicated that when the flange thickness, t_f was maintained as a constant, the ratio negatively affected the shear strength of the LDSS plate girders.

• It was learned that when the web depth was maintained at a constant value, the shear strength increased as the web depth-to-web thickness ratio (h_w/t_w) decreased.

• According to the results of the FEM analysis, when the web depth was maintained at a constant value, the shear resistance of the LDSS plate girders increased with an increase in the ratio of the flange width-to-web depth (b_f/h_w).

• Panels with equal values of web slenderness (h_w/t_w) and having different aspect ratios were observed to possess differing strengths provided by the web contribution. In fact, the higher the values of the aspect ratio, the lesser the contribution of the web panel (χ_w) to the overall resistant mechanism.

• It was noticed that the shear buckling of the LDSS plate girders almost took place during the linear path. According to the geometrical characteristics of the plate girders, the failure in the thin girders was caused by the geometrical nonlinearity, while the failure in the thick girders was attributed to the material nonlinearity.

• A comparison of the failure loads acquired from the FEM analysis with the approximate strengths of the present European Code of Practice, EN 1993-1-4, indicated that the shear strength of the LDSS plate girders had been slightly underestimated by the latter.

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Annex A

Table A1 Required parameters to calculate the design shear resistance according to EN 1993-1-4

Specimen	k_τ^*	λ_w^*	χ_w	M_{Ed} $(KN.mm)$	$M_{f,Rd}$ $(KN.mm)$	c (mm)	c/a
LDSS1G1 $(12-3)$	9.34	2.552	0.421	233,880	550,800	142	0.24
LDSS1G1 $(12-4)$	9.34	1.914	0.524	308,100	550,800	132	0.22
LDSS1G1 $(12-5)$	9.34	1.531	0.613	396,240	550,800	126	0.21
LDSS1G1 $(12-6)$	9.34	1.276	0.693	498,135	550,800	122	0.20
LDSS1G2 $(12-3)$	9.34	2.552	0.421	241,500	734,400	156	0.26
LDSS1G2 $(12-4)$	9.34	1.914	0.524	314,300	734,400	142	0.24
LDSS1G2 $(12-5)$	9.34	1.531	0.613	408,270	734,400	134	0.22
LDSS1G2 $(12-6)$	9.34	1.276	0.693	509,000	734,400	129	0.21
LDSS1G3 $(12-3)$	9.34	2.552	0.421	250,000	918,000	169	0.28
LDSS1G3 $(12-4)$	9.34	1.914	0.524	340,080	918,000	152	0.25
LDSS1G3 $(12-5)$	9.34	1.531	0.613	419,400	918,000	142	0.23
LDSS1G3 $(12-6)$	9.34	1.276	0.693	515,700	918,000	136	0.22
LDSS2G1 $(12-3)$	6.34	3.097	0.360	310,200	550,800	285	0.24
LDSS2G1 $(12-4)$	6.34	2.323	0.453	436,440	550,800	265	0.22
LDSS2G1 $(12-5)$	6.34	1.858	0.535	626,400	550,800	252	0.21
LDSS2G1 $(12-6)$	6.34	1.548	0.609	785,400	550,800	244	0.20
LDSS2G2 $(12-3)$	6.34	3.097	0.360	321,600	734,400	312	0.26
LDSS2G2 $(12-4)$	6.34	2.323	0.453	463,800	734,400	285	0.23
LDSS2G2 $(12-5)$	6.34	1.858	0.535	645,420	734,400	267	0.22
LDSS2G2 $(12-6)$	6.34	1.548	0.609	845,280	734,400	258	0.21
LDSS2G3 $(12-3)$	6.34	3.097	0.360	324,000	918,000	339	0.28
LDSS2G3 $(12-4)$	6.34	2.323	0.453	471,000	918,000	305	0.25
LDSS2G3 $(12-5)$	6.34	1.858	0.535	650,220	918,000	285	0.23
LDSS2G3 $(12-6)$	6.34	1.548	0.609	858,450	918,000	271	0.22

$* k_\tau$ Is the shear-buckling coefficient, λ_w is the web slenderness parameter