Numerical study on duplex stainless-steel plate girder under simultaneous bending and shear

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Abstract. The use of stainless-steel results in significant life-cycling cost savings. It likewise has high-temperature properties, just as satisfactory weldability, and fracture toughness. This study investigates the behaviour of stainless-steel plate girder under simultaneous bending and shear using commercial FE software Abaqus. For modelling stainless steel of grade Duplex EN 1.4462 (2205) was used, which possess twice the design strength of austenitic and ferritic stainless steels. Numerically validated models were used to conduct parametric study covering effect of flange slenderness and flange thickness to web thickness ratios. The results show a significant contribution of the flange on shear strength and bending moment capacity of stainless-steel plate girders. The contribution of the flange was also influenced by the presence of end rigid post and change in the panel aspect ratio.

1. Introduction

As the name indicates, the stainless steel has higher corrosion resistance together with high strength, high stiffness, and high ductility. These characteristics can be exploited in a wide range of construction applications. The stainless-steel materials have more initial cost than carbon steel, but the durability and less maintenance cost make them more economical than carbon steel for constructing structures which have a long life [1]. The steel plate girders are built up I section, used to carry heavy loads which cannot be carried economically by rolled I section. Mainly plate girders are adopted in large-span structures like bridges, industrial power plants etc. The stainless-steel plate girders can effectively utilise a wide range of environments. The behaviour of stainless steel differs from carbon steel due to the absence of a definite yield point in stainless steel. Also, there exists a difference between tension and compression stress-strain data [2].

The numerical modelling of the structural response of stainless-steel hollow sections was done by Gardner, et al. [3]. This study confirmed the ability of Abaqus based FE model to represent the physical stainless steel structural member, covering local and overall buckling behaviour. Rasmussen, et al. [4] developed numerical models for stainless steel plates in compression. Several studies were conducted to investigate the behaviour of stainless-steel plate girder. Among these studies, Real, et al. [5,6,7] investigated the shear behaviour of Austenitic stainless-steel plate girders. Saliba, et al. [8] experimentally and numerically investigated the behaviour of lean duplex stainless steel plate girders and found the importance of considering the effect of rigid end post while developing a design model. Hassanein, et al. [9] checked the behaviour of lean duplex stainless steel girder with slender unstiffened webs. The shear buckling behaviour of stainless-steel plate girder was detailed by Chen, et al. [10]. From the literature it was found that all austenitic and duplex grade plate girder specimens were failed by shear buckling of the web plate, also identified the factors that have effects on the shear buckling.
capacity. Yuan, et al. [11] conducted a further investigation of shear buckling behaviour by providing diagonal stiffeners. Chen, et al. [12] was done an experimental study to examine the effect of bending moment along with shear force on stainless steel plate girder. Maarten Fortan, et.al. [13] carried out an experimental and numerical study on plate girders with non-rigid end post and suggest modification for current Euro code to predict accurate shear capacity. Most of the existing studies fascination on the shear response of plate girder. The shear behaviour studies were focused on the geometrical parameters of the web and neglected the effect of flanges because the web is taking the most of shear stress in I shape flexural members. The flange is effectively resisting the bending moment. This study focuses on the behaviour of stainless-steel plate girder with varying flange size and explore the effect of rigid end post and panel aspect ratio.

2. Numerical modelling

This study was executed by using finite element software Abaqus [14]. The numerical models were developed and validated based on the experimental study conducted by Chen, et al. [10].

2.1. Geometry

The geometry of plate girder is shown in figure 1 and 2 and the geometric dimensions are listed in table 1. The symbol $w_0$ shown in table 1 is represent local imperfection amplitude for each geometry of specimen measured using specially designed setup by X.W Chen[10]. The name of plate girder contains grade of material,nominal web panel depth and aspect ratio. For example, the name V-2205-R500ad1 represent a plate girder specimen made of EN 1.4462 (2205) with nominal web of depth 500 mm and aspect ratio $a/h_w = 1.0$; R indicates the rigid end post.

| Plate girder     | L    | a    | c    | $h_w$ | b    | $t_w$ | $t_f$ | $t_s$ | $w_o$ |
|------------------|------|------|------|-------|------|-------|-------|-------|-------|
| V-2205-500ad1    | 1198.3 | 499.0 | 100.2 | 498.4 | 150.2 | 3.90  | 12.59 | 12.59 | 0.75  |
| V-2205-500ad1.5  | 1698.3 | 748.7 | 100.4 | 499.4 | 150.1 | 3.90  | 12.59 | 12.59 | 1.24  |
| V-2205-R500ad1   | 1198.9 | 498.7 | 100.8 | 498.9 | 150.1 | 3.90  | 12.59 | 12.59 | 1.19  |

All dimensions are in mm

![Figure 1. Plate girder with non-rigid end post](image1.png)

![Figure 2. Plate girder with the rigid end post](image2.png)

2.2. FE modelling

Widely used four-node doubly curved general-purpose S4R shell element was used to model the plate girder. The element size was taken as 15x15 mm. simply supported three-point bending setup was used to test the plate girder. The load was applied at the mid-span. The boundary conditions were taken in the FE model as shown in figure 3, where the symbol $u_x, u_y$ and $u_z$ represent displacements along with x, y and z axes, respectively, while the $\theta_x, \theta_y$ and $\theta_z$ are the corresponding rotations.
2.3. Material modelling

The stainless steel of grade Duplex EN 1.4462 (2205) was used in this study. Material properties obtained from tensile coupon tests conducted by Yuan, et al. [11] were used to define the material model, listed in Table 2. The material was modelled by using two-stage modified Ramberg-Osgood expressions proposed by Gardner and Ashraf[15] given by equation 1 and 2. Where $n$ and $n_{0.2,1.0}$ are strain hardening exponents. $E_{0.2}$ is the tangent modulus at 0.2% offset strain given by equation (3). The stress-strain model was converted into true stress versus true strain form which was used to define the material.

$$
\varepsilon = \frac{\sigma}{E_0} + 0.002 \left( \frac{\sigma}{\sigma_{0.2}} \right)^n \left( \sigma < \sigma_{0.2} \right) \quad (1)
$$

$$
\varepsilon = \frac{\sigma - \sigma_{0.2}}{E_{0.2}} + \left( \varepsilon_{1.0} - \varepsilon_{0.2} - \frac{\sigma_{1.0} - \sigma_{0.2}}{E_{0.2}} \right) \times \left( \frac{\sigma - \sigma_{0.2}}{\sigma_{1.0} - \sigma_{0.2}} \right)^{n_{0.2,1.0}} + \varepsilon_{0.2} \left( \sigma \geq \sigma_{0.2} \right) \quad (2)
$$

$$
E_{0.2} = \frac{\sigma_{0.2}E}{\sigma_{0.2} + 0.002nE} \quad (3)
$$

Where $\sigma_{0.2}, \sigma_{1.0}, \varepsilon_{0.2},$ and $\varepsilon_{1.0}$ are proof stress at 0.2%, proof stress at 1%, total strain at $\sigma_{0.2}$ and total strain at $\sigma_{1.0}$ respectively.

### Table 2. Average measured material properties from tensile coupon tests (11)

| Grade | t (mm) | $E_0$ (MPa) | $\sigma_{0.01}$ (MPa) | $\sigma_{0.2}$ (MPa) | $\sigma_{1.0}$ (MPa) | $\sigma_u$ (MPa) | $\varepsilon_u$ (%) | $\varepsilon_f$ (%) | $n$ | $n_{0.2,1.0}$ |
|-------|--------|-------------|------------------------|----------------------|---------------------|-----------------|------------------|------------------|----|----------------|
| 1.4462 | 3.90   | 204,800     | 345.3                  | 539.6                | 604.1               | 761.4           | 26.1             | 40.2             | 6.7 | 3.3            |
|       | 12.59  | 184000      | 227.8                  | 464.6                | 552.8               | 705.3           | 23.3             | 37.4             | 4.2 | 4.4            |

**Figure 3.** Boundary conditions
2.4 Analysis
Initially, a linear eigenvalue buckling analysis was performed to generate the lowest relevant elastic buckling mode. The elastic buckling mode utilised as the initial geometric imperfection shape. Before submitting the buckling analysis job, the node coordinates for the different mode shapes need to be written as output. This output was consequently introduced as imperfections for the non-linear analysis to achieve a smoother transition to the post-buckling region. For this *IMPERFECTION command available in Abaqus was used and the measured amplitude of imperfections was incorporated for FE analysis. The modified Riks method available in the Abaqus was used for non-linear analysis, enabling post-buckling analysis of the plate girder specimen.

2.5 Validation
The precision of the FE models was evaluated by comparing the load versus mid-span displacement curves, ultimate shear resistances and shear buckling failure modes against physical experiment conducted by X.W. Chen, et.al [10]. The force versus mid-span vertical displacement curves of three different models V2205-500ad1, V2205-500ad1.5, and V2205-R500ad1 are shown in figure 4. The comparison of ultimate shear resistance between experiment and FE models listed in Table 3. The average ratio of FE predicted ultimate shear resistances to test values is 1.007. The comparison of the failure modes of model V-2205-500ad1 is shown in figure 5. From the results, it is confirmed that the numerical models show good agreement with the experimental model.

![Figure 4](image1.png)

![Figure 5](image2.png)

Table 3. Comparison of ultimate shear resistance between test [10] and FE results

| Plate girder       | \( V_{u,\text{Test}} \) (kN) | \( V_{u,\text{FE}} \) (kN) | \( \frac{V_{u,\text{FE}}}{V_{u,\text{Test}}} \) | Error (%) |
|-------------------|-------------------------------|----------------------------|---------------------------------|-----------|
| V-2205-500ad1     | 453.9                         | 462.64                     | 1.02                            | 1.92%     |
| V-2205-500ad1.5   | 385.9                         | 385.68                     | 0.99                            | -0.057%   |
| V-2205-R500ad1    | 512.7                         | 516.46                     | 1.01                            | 0.73%     |

![Figure 4](image3.png)
2.6. Parametric study
The numerically validated models were used to investigate further detailed parametric study. The main parameters focused on this study were web slenderness \((h_w/t_w)\) by varying web thickness \((t_w)\), flange slenderness \((b_f/t_f)\) by varying flange thickness \((t_f)\) and flange thickness to web thickness ratio \((t_f/t_w)\) by varying both web thickness and flange thickness. Stiffener thickness \((t_s)\) = 12.56 mm is kept constant for all models.

3. Results and discussions
To understand the contribution of flanges in a stainless-steel plate girder extensive parametric study was done. The numerical results are plotted in the form of shear force and bending moment. The results are summarized in the following sections.

3.1. Effect of \(t_f/t_w\)
The flange thickness to web thickness ratios varies from 1 to 4 and the behaviour was analysed for varying flange slenderness from 75 to 150. Figure 6 depicts the influence of flange thickness to web thickness ratio on the shear force. A large value of shear force was obtained at \(t_f/t_w=4\) for all flange slenderness from 75 to 150. With an increase in \(t_f/t_w\) from 1 to 2, up to 35% of the increase in shear force was observed in model with panel aspect ratio 1.5(V2205-500ad1.5). For the same model shear force increase was only up to 11% when \(t_f/t_w\) is going from 2 to 4. When the thickness of the web is less (higher web slenderness) the end rigid post helps to increase the shear capacity. This could be observed in figure 6(d) where the shear capacity of model V-2205-R500ad1 was around 1.18 times the shear capacity of model V-2205-500ad1. It indicates the ability to end rigid post to increase the shear capacity.

![Figure 6](image_url)

**Figure 6.** Shear force verses \(t_f/t_w\) (a) web slenderness=75, (b) web slenderness=100, (c) web slenderness=128, (d) web slenderness=150
3.2. Effect of flange slenderness \( b_f / t_f \)

Effect of \( b_f / t_f \) is graphically present in Figure 7 which is examined for different web slenderness over a range of \( h_w / t_w = 75-150 \) by varying \( t_w \), keeping \( h_w \) as constant. For flange slenderness \( b_f \) kept constant and set \( t_f = 1-4 \) times \( t_w \). The percentage shear capacity reduction observed in models V2205-500ad1, V2205-R500ad1 and V2205-500ad1.5 were up to 17\%, 30\% and 33\% respectively. The variation of flange slenderness is highly influenced by rigid end post and high panel aspect ratio. In the case of plate girders with lower web slenderness value, the effect of end rigid post was not significant. Whereas in case of plate girders with higher web slenderness value, the presence of end rigid post increases the shear capacity at same flange slenderness which is similar to the trend observed in the previous section (3.1).

![Figure 7](image1.jpg)  
**Figure 7.** Shear force versus the flange slenderness, (a) web slenderness=75, (b) web slenderness=100, (c) web slenderness=128, (d) web slenderness=150.

3.3. Effect of bending moment on flange slenderness \( b_f / t_f \)

The variation of bending moment versus flange slenderness is shown in figure 8. All girders with low flange slenderness ratios show better bending moment resistance. It is interesting to notice that the bending moment resisted by three models at higher slenderness shows almost similar value. This is because at higher flange slenderness (low web thickness) the flange is insufficient to resist bending moment. At this stage, the overall bending moment resistance depends upon the web contribution, which is almost equal for all models. The percentage bending moment resistance reduction observed in models
V2205-500ad1, V2205-R500ad1 and V2205-500ad1.5 were up to 17%, 30% and 33% respectively. The numerical results show that the bending moment resistance of girder with end rigid post subjected to higher web slenderness and lower flange slenderness was up to 1.17 times the bending moment resistance by girder without end rigid post.

![Graphs showing bending moment versus flange slenderness](image)

**Figure 8.** Bending moment versus flange slenderness (a) web slenderness=75, (b) web slenderness=100, (c) web slenderness=128, (d) web slenderness=150.

### 4. Conclusions

This study intends to understand the behaviour of stainless-steel plate girder under the action of simultaneous bending and shear. Three numerical models were validated against the experimental work by Chen, et al [10]. Within the limited parametric study, the following conclusions were obtained:

- The shear capacity of plate girder increases with an increase in $t_f/t_w$.
- Providing the end rigid post and increasing flange thickness helps to reduce web thickness without affecting the shear resistance.
- The variation of flange slenderness is highly influenced in plate girder with rigid end post and higher panel aspect ratio.
- Girders with low flange slenderness ratios show better bending moment resistance.
- Plate girder with end rigid post can resist more bending moment than plate girder without end rigid post.
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