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Shear strength of dual web plate girders with transverse stiffeners

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ABSTRACT

Although conventional I-section plate girders are widely used, they are still considered as open section girders; thus, lacking the adequate torsional stiffness. This causes them to be vulnerable to fail due to lateral torsional/distorsional buckling. One of the methods of improving that torsional stiffness is to turn the cross-section into a closed one. Although it is common in literature to replace the flat flanges with tubular ones, another approach is followed herein through adding additional web to the girder. Therefore, a numerical study is conducted in this research, using Ansys Workbench, to investigate the shear behaviour of transversely stiffened dual web plate girders (DWPGs). In the adopted finite element (FE) model, each of initial geometric imperfections, geometrical and material nonlinearities are taken into account. A parametric study is carried out on simply supported DWPGs involving stiffeners arrangement, web panel aspect ratio and web spacing. It is found that equipping the DWPGs with inner stiffeners between their dual webs is preferable to outer stiffeners. Particularly, DWPGs with inner stiffeners have higher shear strength and ductility compared to other DWPGs.

Keywords: Dual web plate girder; Shear strength; Finite element

1. Introduction

Conventional I-section plate girders, shown in Fig. 1, commonly consists of two flanges and a web, which are all welded together, in addition to transverse stiffeners. Main role of the flanges is to resist the acting bending moment, while the web is responsible for resisting the applied shear force. It is common practice that the induced shear forces are lower than the axial forces, that results from the bending moment, acting on the flanges. Thereupon, flanges are usually fabricated from plates that are thicker than those of the web. Anyway, those I-section plate girders can fail in several modes such as web shear buckling, flexural-torsional buckling, compression flanges local buckling, etc. Following the design guidelines of different codes of practices can hinder those failure modes. On the other hand, flexural-torsional buckling of I-section plate girders can be disallowed by turning the open section of the plate girder into a closed one [1–3]. In this research, additional web is proposed to be installed parallel to the original one, as shown in Fig. 1, to turn the whole plate girder into a dual web plate girder (DWPG) that has a closed cross-section. Transverse stiffeners of the DWPGs may be placed in the inner region between the webs (Fig. 2a) or on their outer surface (Fig. 2b). However, the process of the web design is involved with attaining a sufficient web thickness accompanied with an optimum stiffener spacing [4].

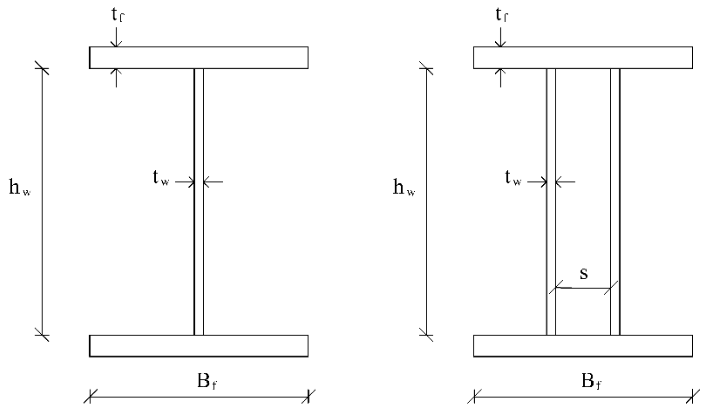
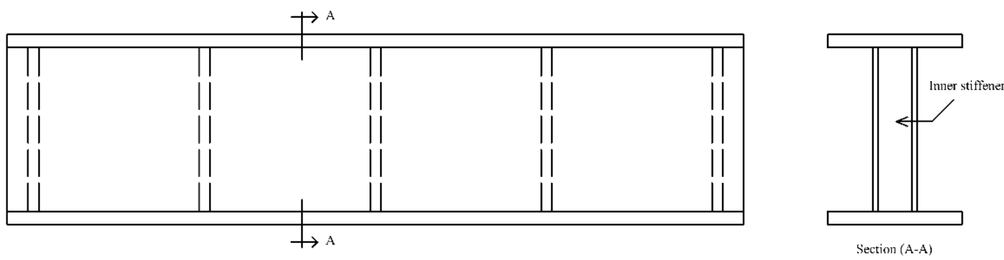
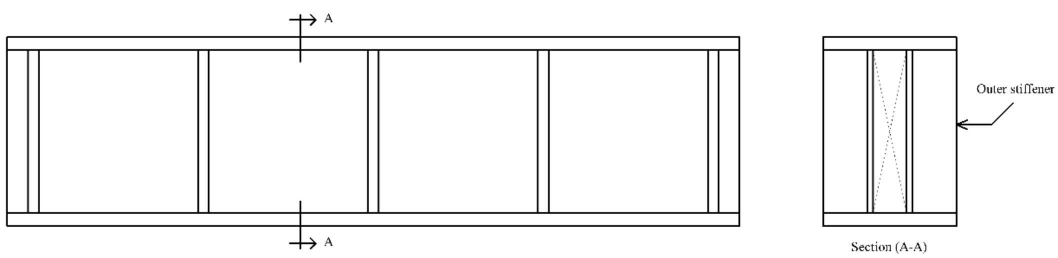


Fig. 1. Conventional I-section plate girder (left) and DWPG (right).



(a) with inner stiffeners



(b) with outer stiffeners

Fig. 2. Typical DWPG.

In spite of the development of the diagonal tension field theory by Wagner in 1931 [5], the elastic buckling approach has been still used for the design of plate girders till recently. Later, distinctive approaches were developed to determine the ultimate shear capacity of steel plate girders [6,7].

The finite element (FE) results found in literature [8] demonstrate that the forming of plastic hinges in flanges may either rely on a flexural or shear base. In the bending-initiated mechanism, plastic hinges develop at mid-spans; whereas in the shear-initiated mechanism, plastic hinges only develop in the end panels. If flange plates are not adequately rigid to resist bending-based normal stresses, plastic hinges develop at the position of maximum bending moment.

Subsequent to web shear buckling, the plate becomes unable to bear extra compressive stresses; thus, a different load resistance mechanism develops, though which any sort of extra shear loading is supposed to be carried by an inclined tensile stress field. Moreover, the tensile membrane stress keeps expanding, till reaching the yield stress of the material, as the acting loading increases. When the web becomes fully yielded, eventual collapse will occur. This particularly happens as a result of the formation of the plastic hinges in the flanges which cause a shear sway failure mechanism [7,9].

Alternatively, design of plate girders to resist shear was under consideration in several studies [8–12]. Olsson [13] derived design formulas for shear strength of I-section plate girders. Shanmugam et al. [14] studied the load carrying capacity of curved plate girders both experimentally as well as numerically. Alex et al. [15] numerically investigated buckling of DWPGs.

The preceding brief survey of research information regarding shear behaviour of plate girders illustrates that there is no sufficient work have been conducted to evaluate DWPGs shear behaviour. Consequently, this research is concerned with shear strength and behaviour of DWPGs installed with transverse stiffeners. Hence, different parameters (e.g., stiffeners arrangement, web panel aspect ratio and web spacing) effect is taken into consideration in the current research.

2. Numerical modelling and validation

For the sake of evaluating shear behaviour of DWPGs, an FE analysis was carried out herein on full-scale plate girders. The main reason for not adopting the isolated web panel approach is that it does not efficiently predict plate girders web plates behaviour [7]. Accordingly, Ansys Workbench (2020 R1) [16] was utilized for that purpose. Eight FE models of DWPGs, with different parameters, were modelled in the latter software, thus, their shear behaviour could be assessed. The key parameters were selected, in this research, based upon the survey besides previous experience in the topic under consideration, since both authors carried out numerous studies dealing with the shear strength of structural elements; refer to References [1,17–19]. In consequence, the most logical parameters that were expected to influence the shear performance of DWPGs are as follows:

1) stiffener arrangement; (inner or outer), as previously shown in Fig. 2,
2) web panel aspect ratio (a/h_w); (0.5 and 1.0), and
3) web spacing ratio-to-web height (s/h_w); (0.10 and 0.25).

The span length, webs height, webs thickness, flanges width and flanges thickness of all DWPGs, under consideration, were fixed to 4000, 1000, 4, 350 and 16 mm, respectively. In addition, transverse stiffeners were used, and their thickness was chosen to be 20 mm through the whole FE program. As depicted in Fig. 3, the DWPGs were loaded, with a mid-span concentrated monotonic load, as being simply supported with non-rigid end posts having edge distance e of 100 mm. Table 1 lists the main differences between the specimens of the current research. It is worth to note that the specimens were labelled such that their characteristics could be identified from their label. In detail, each label starts with either "GO: which refers to "girder with outer stiffeners" or "GI" which refers to "girder with inner stiffeners"; it is then followed by two numbers separated by $(-)$. The first number represents the web panel aspect ratio a/h_w, whereas the second refer to web spacing ratio to web height s/h_w.

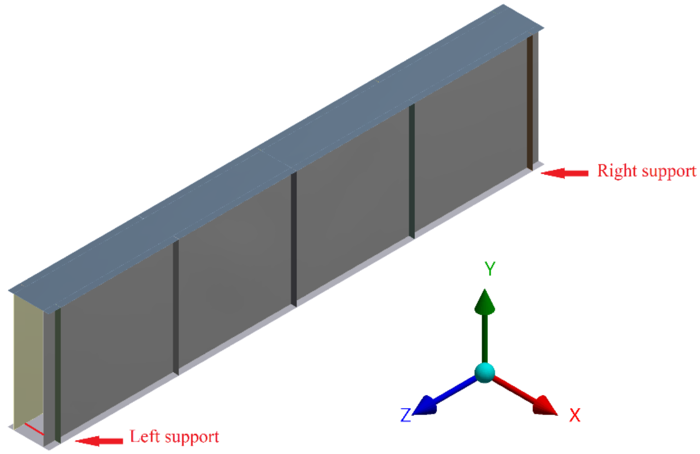


Fig. 3. FE model of a typical DWPG.

Table 1. Details of the specimens.

Specimen	Stiffener arrangement	a/h_w *	s/h_w *
GO-0.5-0.10	outer	0.5	0.10
GO-1.0-0.10	outer	1.0	0.10
GO-0.5-0.25	outer	0.5	0.25
GO-1.0-0.25	outer	1.0	0.25
GI-0.5-0.10	inner	0.5	0.10
GI-1.0-0.10	inner	1.0	0.10
GI-0.5-0.25	inner	0.5	0.25
GI-1.0-0.25	inner	1.0	0.25

* $h_w = 1000\ mm$

The type of FE analysis carried out herein was the common Load-displacement nonlinear analysis. Based on this particular type of analysis, the ultimate loads, failure mechanisms and response of DWPGs can be obtained. In order to accurately capture the nonlinear behaviour of DWPGs, initial geometrical imperfections should be taken into account. Thereby, they were taken herein following the guidelines of EN 1993-1-5 [20]; refer to Appendix C.5 of the latter reference. Despite the slight variation of the initial geometrical imperfection between each specimen, it does not surpass the maximum value allowed by the Bridge Welding Code (21); $h_w/100$. With regard to the residual stresses, they were neglected, as recommended by Dong and Sause (22), since the unbraced lengths of all of the proposed specimens were less than 20 m.

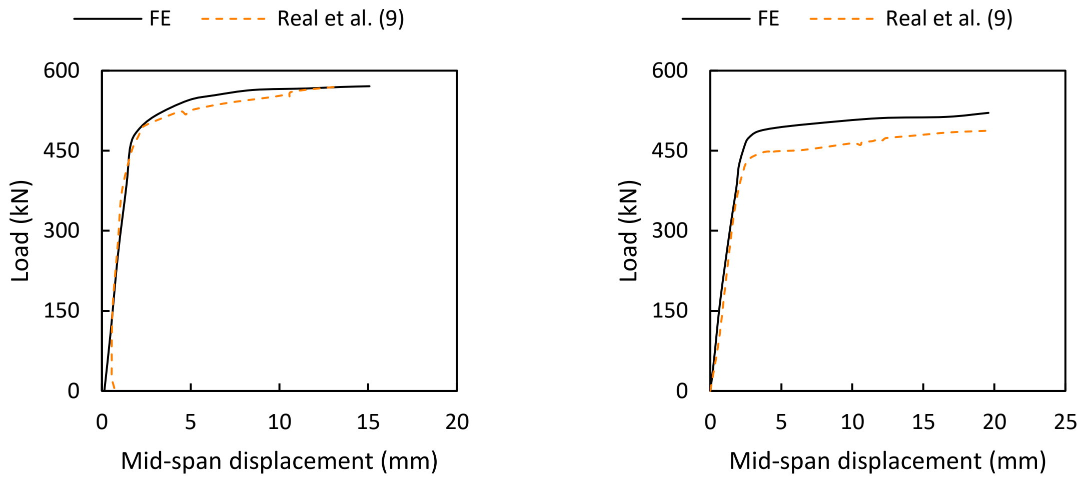
Due to the fact that DWPGs were composed of thin-walled plates, shell elements were used to mesh the FE model. So, the four node SHELL181 element is employed in the current research. Each node of that element possesses six degrees of freedom. Moreover, a mesh sensitivity analysis (convergence test) should be conducted in exchange for assessing the requirement of a mesh refinement. Yet, Lee et al. [4] and Alinia et al. [7] recommended to discretise the web height into 16 to 30 elements. Subsequently, the web was decided to be discretised into an intermediate value of 20 elements in this research. For the dimensions of the current specimens, a mesh size of 50 mm was consequently selected. The nonlinear geometry was taken into account through turning on the “Large displacements” option in Ansys Workbench [16].

The left and right supports, located at an edge distance e on the bottom flange as shown in Fig. 3, were free to translate in z-direction and to rotate about x-axis. Other translations and rotations were obviated. At the mid-span of the DWPG, a centre point at the lower flange was obviated to translate in z-direction. Thus, the loading application mechanism and end support restraints ensured constant shear stress distribution in all web panels.

The steel material was modelled as a von-Mises material with isotropic hardening. The used steel was S355 which has a yield and an ultimate strength of 355 and 510 MPa, respectively, as per EN 1993-1-1 [23]. The stress-strain curve of the adopted steel was defined as a bilinear curve with linear strain hardening, having a post-yield tangent modulus of 1% of the material’s Young’s modulus, as recommended in Appendix C.6 of EN 1993-1-5 [20]. As to the elastic properties of the utilized steel, it has a Young’s modulus of 200,000 MPa and a Poissoin’s ratio of 0.3.

Shear behaviour of DWPGs, under consideration, have not been extensively investigated in literature. Thereby, the conclusions of the current FE study would raise questions and suspects unless sufficient verification studies are conducted to validate the adopted FE models. Accordingly, the validation was carried out herein by simulating the experimental testing of Real et al. [9].

Real et al. (9) experimentally and numerically tested nine I-section plate girders to evaluate their shear response. However, only two of these specimens (ad1w4 and ad2w4) were selected for the current verification study. This is because Real et al. [9] did not clearly provide the experimental load-deflection curves of the remaining specimens. Both specimens, under consideration, shared some properties. Particularly, they had identical web height of 500 mm, identical web thickness of 4 mm, and identical flanges and stiffeners thickness of 20 mm. The main differences between the three specimens were their span length and stiffener spacing. In details, the span lengths of ad1w4 and ad2w4 were 1 and 2 m, respectively, whereas stiffener spacings were 500 and 1000 mm, respectively. It should be mentioned that the flange width and edge distance of the plate girders were not neatly reported in the paper. Anyway, they were assumed herein to be equal to 250 mm and 100 mm, respectively. The two specimens were fabricated from austenitic stainless-steel grade 1.4301 (AISI 304). This grade of stainless-steel has a Young’s modulus of 197240 MPa and 187340 MPa for 4 mm and 20 mm thick plates, respectively. Regarding Poisson’s ratio, it is equal to 0.3. The stress-strain curve of that stainless-steel was clearly provided by Real et al. [9]; thus, it was modelled herein utilizing multilinear isotropic hardening model. The specimens were loaded as being simply supported beams with mid-span concentrated load; therefore, the boundary conditions discussed earlier in this section were adopted. The mid-span concentrated load was applied at the intersection of the mid-span transverse stiffener and the upper flange. This is to avoid excessive local deformations of the flange; see Reference [3]. The load-deflection curves obtained from the experimental test of Real et al. [9] as well as those of the current FE modelling are plotted together in Fig. 4. It is evident that the experimental and FE curves match each other in terms of initial stiffness, post-yield stiffness and ultimate strength. The ratio between the FE ultimate load and the experimental ultimate load for specimens ad1w4 and ad2w4 was found to be 1.03 and 1.0, respectively. Thereupon, the current FE models proved their accuracy in predicting the shear strength of plate girders. This enabled the initiation of a comprehensive parametric study on DWPGs with different characteristics.



(a) ad1w4 (b) ad2w4

Fig. 4. Load-deflection curves of Real et al.'s (9) specimens.

3. Results and discussions

The main objective of this research is to investigate the shear behaviour of transversely stiffened DWPGs made of S355 steel. So, the ultimate shear strength V_{FE} of the specimens, under consideration are listed in Table 2. Moreover, mechanism of failure of the specimens are also reported in this section. In order to get a deep insight into the shear strength results, the plastic shear strength V_P of a DWPG was assumed to be equal to the plastic shear resistance of the webs only; hence, it can be determined as follows (8):

$$V_P = A_V \sigma_y / \sqrt{3} \tag{1}$$

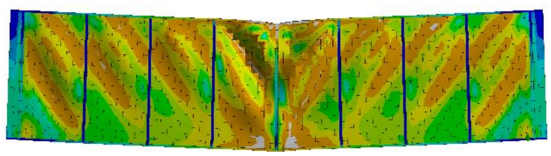
where A_V is the shear area of the dual webs ($= 2h_w t_w$) and σ_y is the material tensile yield stress.

Table 2. Shear strengths of the specimens.

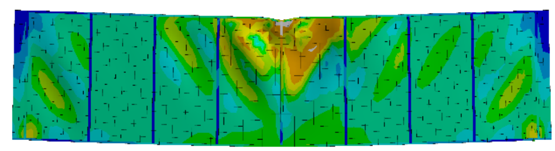
Specimen	$a/h_w{}^*$	$s/h_w{}^*$	V_{FE} (kN)	V_{FE}/V_P
GO-0.5-0.10	0.5	0.10	801	0.49
GO-1.0-0.10	1.0	0.10	682	0.42
GO-0.5-0.25	0.5	0.25	890	0.54
GO-1.0-0.25	1.0	0.25	706	0.43
GI-0.5-0.10	0.5	0.10	1008	0.61
GI-1.0-0.10	1.0	0.10	745	0.45
GI-0.5-0.25	0.5	0.25	1132	0.69
GI-1.0-0.25	1.0	0.25	819	0.50

$^* h_w = 1000\ mm$

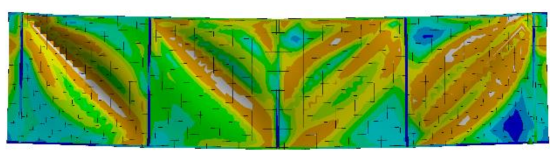
Fig. 5 shows the von-Mises stress distribution at V_{FE} for the DWPGs under consideration. It is worth mentioning that the white contours represent the regions which exceeded the tensile yield stress of the material (355 MPa). All the specimens exhibited local buckling of the upper flanges. This is attributed to the applied mid-span concentrated load. Also, that local buckling surely influenced the von-Mises stress distribution causing it to be unsymmetric. Moreover, plastic hinges were found to develop in the upper flanges of all specimens due to that local buckling. Despite this, all the specimens also exhibited considerable shear buckling in their webs. Thereupon, the specimens may be considered to fail due to the interaction of shear and flexural plastic hinges. That local buckling of the flanges may have been hindered through equipping the DWPGFs with thicker full-width stiffeners at their mid-spans. In general, the specimens with outer stiffeners exhibited more obvious web shear buckling compared to those with inner stiffeners. This goes to the lower width of the outer stiffeners compared to the inner ones. Additionally, the specimens with higher web panel aspect ratio ($a/h_w = 1.0$) had more regions of their web panels exceeding the yield strength compared to those with lower web panel aspect ratio ($a/h_w = 0.5$). It can also be detected, from the stress distribution plots, that no shear-based plastic hinges were developed in the flanges of the DWPGs. This is opposite to the traditional I-section plate girders mode of failure].



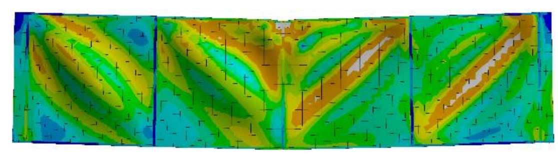
(a) GO-0.5-0.10



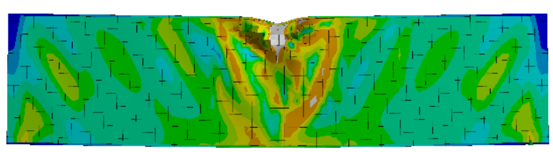
(b) GO-1.0-0.10



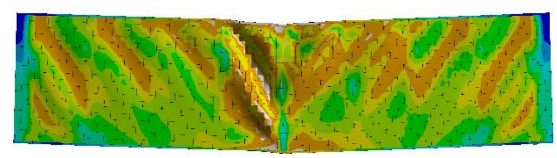
(c) GO-0.5-0.25



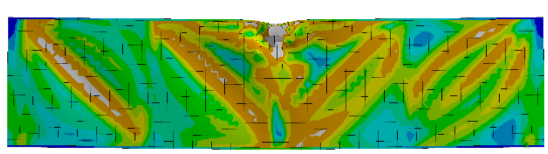
(d) GO-1.0-0.25



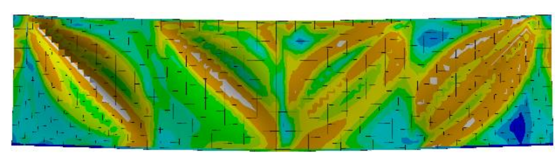
(e) GI-0.5-0.10



(f) GI-1.0-0.10



(g) GI-0.5-0.25



(h) GI-1.0-0.25

Fig. 5. A plot of von-Mises stress distribution at V_{FE}.

Stiffeners' arrangement significantly affected the shear strength of DWPGs. Referring to the bar chart in Fig. 6, it is seen that DWPGs with inner stiffeners generally had higher V_{FE}/V_P ratio compared to those with outer stiffeners. This is attributed to the fact that inner stiffeners link both webs together; thus, forcing them to act as a single unit in resisting shear. In addition, the average V_{FE}/V_P ratio for the specimens with outer stiffeners was found to be 0.44. On the other hand, the same average ratio for the specimens with inner stiffeners was found to be 0.56. Thus, utilizing inner stiffeners, in lieu of outer ones, in DWPGs could enhance their shear carrying capacity by about 27%. Moreover, Fig. 6 shows the load-deflection curves of GO-0.5-0.10 and GI-0.5-0.10. As detected from their label, they both shared the same characteristics and geometric dimensions except for stiffeners arrangement. Except for the ultimate load, the response of both specimens was nearly identical. Both of them exhibited softening behaviour subsequent to the step in which they reached their ultimate carrying capacity. Nevertheless, GO-0.5-0.10 (with outer stiffeners) had a relatively longer hardening plateau compared to GI-0.5-0.10 (with inner stiffeners). Yet, the latter one surpassed the first in terms of shear strength.

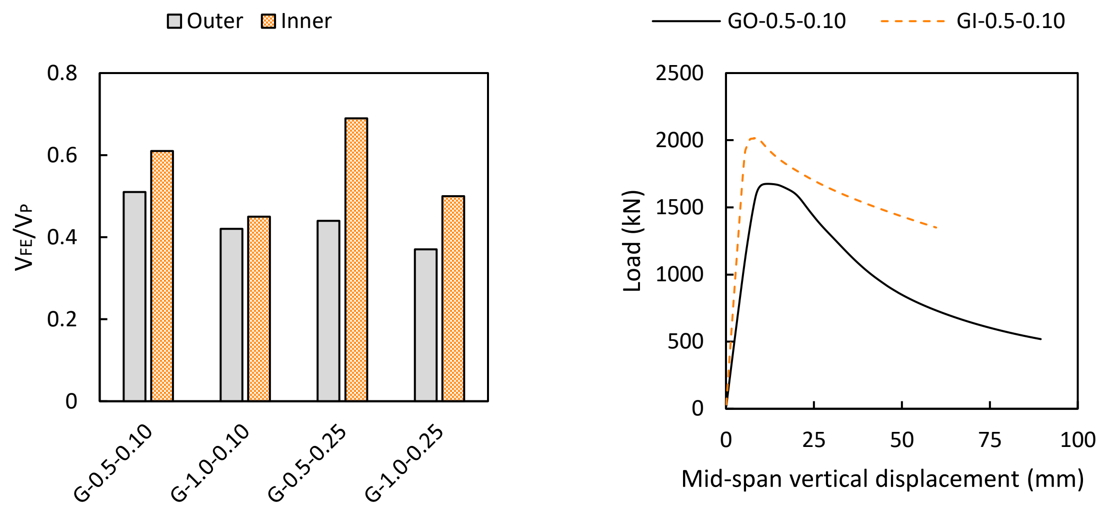


Fig. 6. Effect of stiffeners arrangement on the shear strength of DWPGs.

The bar chart in Fig. 7 represents the V_{FE}/V_P ratios for different web panel aspect ratios a/h_w. It is clear that DWPGs with lower web panel aspect ratios can attain higher shear strength, and this does make sense. The reason is that lowering the web panel aspect ratio a/h_w, while fixing the web height, causes the stiffeners spacing to decrease; this, on its own, enhances the web resistance to out-of-plane buckling. Also, in the same figure, the load-deflection curves of GO-0.5-0.25 and GI-1.0-0.25 are plotted. For those, particular specimens, doubling the web panel aspect ratio a/h_w (from 0.5 to 1) caused the shear strength to drop by 16%. Additionally, the initial stiffness of the DWPGs was found to be inversely proportional to the web panel aspect ratio. The reason is that the specimen with lower web panel aspect ratio was equipped with more stiffeners than the one with higher web panel aspect ratio. Those more stiffeners, for sure, resulted in an increase of the initial stiffness of the DWPGs.

The web spacing ratio-to-web height s/h_w was one of the key parameters that had a weird effect on the shear strength of HTFPGs. Referring to the bar chart in Fig. 8, it is obvious that if and only if the stiffeners were placed out of the dual webs, the s/h_w ratio became inversely proportional to the shear strength. On the other hand, s/h_w ratio was directly proportional to the shear strength of DWPGs with inner stiffeners. To make this clear, increasing s/h_w ratio for DWPGs with outer stiffeners caused the inner distance between the dual webs to increase; thereby, decreasing the outer distance till the edge of the flanges. Consequently, the stiffeners width got smaller which certainly affected the shear strength of the DWPGs in a negative manner. On the opposite, increasing s/h_w ratio for DWPGs with inner stiffeners resulted in an increase of the width of the stiffeners between the webs; thereby, improving the shear strength of the girders. In Fig. 9, the load-deflection curves of GO-1.0-0.10, GO-1.0-0.25, GI-1.0-0.10 and GI-1.0-0.25 can be viewed. It can be detected that s/h_w ratio did not have any considerable effect on the initial stiffness of DWPGs. For GO-1.0-s/h_w specimens, increasing the web spacing ratio-to web-height from 0.1 to 0.25 caused the shear strength to drop by 12%. However, load-displacement response of the girders for both s/h_w ratios was nearly identical. On the contrary, increasing s/h_w ratio, two times and half, for the specimens with inner stiffeners caused the shear strength to increase by 11%. Moreover, in the case of DWPGs with inner stiffeners, increasing s/h_w enhanced the softening behaviour of the girders after they reach their ultimate load capacity.

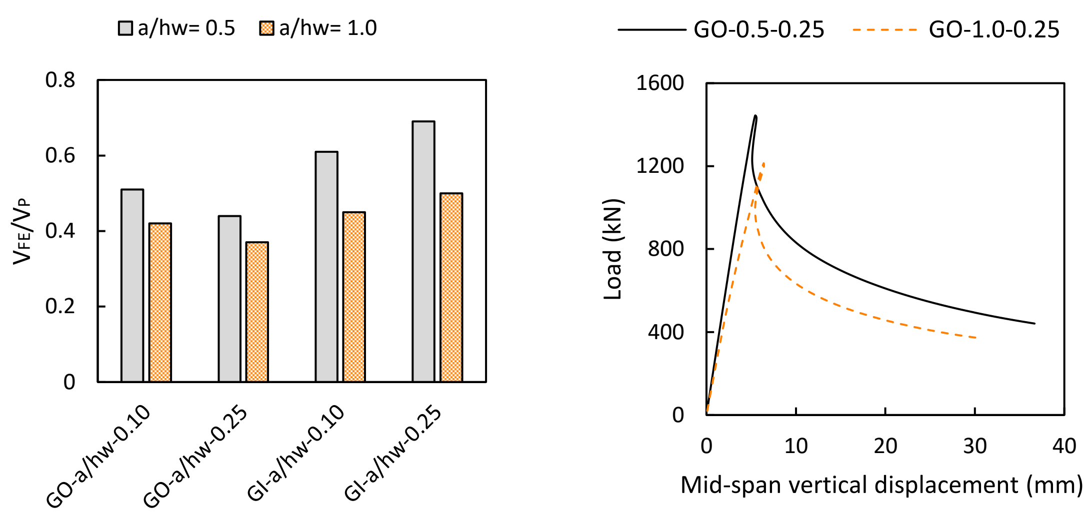


Fig. 7. Effect of web panel aspect ratio on the shear strength of DWPGs.

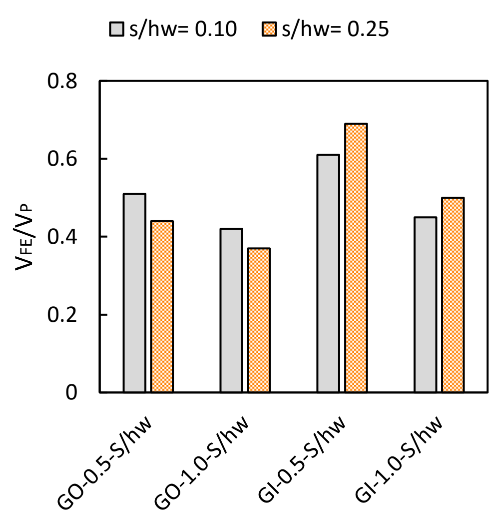


Fig. 8. Effect of web spacing ratio-to-web height on the shear strength of DWPGs.

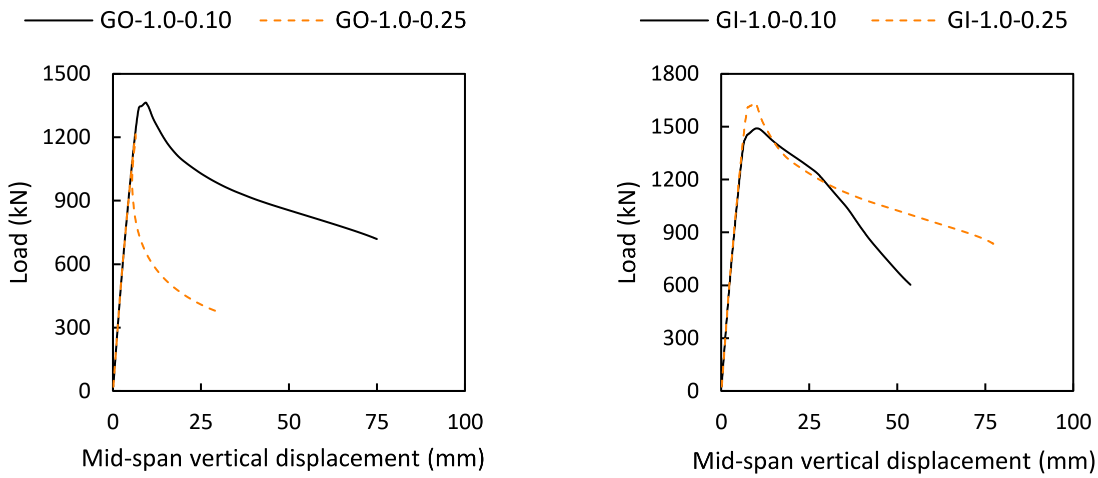


Fig. 9. Load-deflection curves for girders GO-1.0-s/h_w and GI-1.0-s/h_w.

To evaluate the behaviour of the DWPGs after they reach their ultimate carrying capacity and before starting to tear, the virtual ductility index μ_v should be determined. This index is a dimensionless one which is used to measure the deformability of structural elements till their failure. That virtual ductility index can be determined as the following:

$$\mu_v = \delta_{0.85V_{FE}}/\delta_{V_{FE}} \tag{2}$$

where $\delta_{V_{FE}}$ is the displacement at which the DWPG reach its ultimate shear strength while $\delta_{0.85V_{FE}}$ is the displacement at which the DWPG's shear strength drops by 15% after reaching its ultimate value. Virtual ductility indices for all the specimens are described in Table 3. Girders with outer stiffeners had an average virtual ductility index of 1.47 whereas those with inner stiffeners had an average one of 2.44. Accordingly, equipping DWPGs with inner stiffeners did not only enhance their shear strength but they also improved their virtual ductility by about 66%. Of course this represents a great enhancement because the higher the ductility is, the more energy the girder can sustain before failing; see Reference [1].

Table 3. Virtual ductility index of the specimens.

Specimen	$\delta_{0.85V_{FE}}$ (mm)	$\delta_{V_{FE}}$ (mm)	μ_v
GO-0.5-0.10	25.27	11.83	2.14
GO-1.0-0.10	15.91	9.29	1.71
GO-0.5-0.25	5.50	5.46	1.01
GO-1.0-0.25	6.48	6.39	1.01
GI-0.5-0.10	24.0	8.58	2.80
GI-1.0-0.10	25.29	10.02	2.52
GI-0.5-0.25	32.46	11.61	2.80
GI-1.0-0.25	15.70	9.51	1.65

4. Concluding remarks

One of the methods to enhance torsional stiffness of an opened cross-section is to turn it into a closed one. Therefore, dual web plate girders (DWPGs) may be used instead of conventional I-section plate girders for that particular reason. However, no great attention have been pointed out in literature to the shear strength and behaviour of those DWPGs. Accordingly, the current research focused on the assessment of the shear response of DWPGs with transverse stiffeners. All the girders in this research were numerically analysed through the finite element (FE) software Ansys Workbench (2020 R1) [16]. Based on the results of the FE models, the following conclusions could be drawn:

1) The proposed specimens in this research failed due to the interaction of both web shear buckling and flexural plastic hinges development.
2) Equipping DWPGs with inner stiffeners is more recommended than equipping them with outer ones. This reflects on the shear strength of the girders under consideration. In general, DWPGs with inner stiffeners had higher shear strength than those of DWPGs with outer ones by about 27%.
3) Decreasing the web panel aspect ratio had a positive effect on the shear strength of DWPGs.
4) In the case of DWPGs with outer stiffeners, increasing the web spacing ratio-to web height caused the shear strength to be reduced. In contrast, DWPGs with inner stiffeners had their shear strengths increased when the web spacing ratio-to web height was increased.
5) DWPGs with inner stiffeners were more ductile than those with outer stiffeners by an average of 66%. Thus, they can withstand larger deformations, before tearing, when acted upon by loads.

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