

A New Approach to Estimate the Critical Shear Stress of Stiffened Plate Girder

Asmaa Y Hamed (✉ asmaa_yasseen@yahoo.com)

Higher Institute of Engineering and Technology Luxor <https://orcid.org/0000-0001-7881-8601>

Research Article

Keywords: Plate Girder, Stiffened Web, Shear Strength, Minimum Potential Energy, Ansys

Posted Date: February 28th, 2022

DOI: <https://doi.org/10.21203/rs.3.rs-1404273/v1>

License:  This work is licensed under a Creative Commons Attribution 4.0 International License.

[Read Full License](#)

A New Approach to Estimate the Critical Shear Stress of Stiffened Plate Girder

Asmaa.Y.Hamed^{*a}, Amr.B.Sedeek^b

^a Structure Engineering Department, The High Institute of Engineering and Technology Luxor, Egypt
Email: Asmaa_yasseen@yahoo.com

^b Civil Engineering Department, Beni-suef University, Egypt. Email: amrbkr93@gmail.com

Abstract

Despite years of studies on critical buckling stresses, choosing the perfect boundary conditions at the junction between the web plate and flange & stiffener is still a question of engineering judgment. A conservative design criterion is adopted by all current codes of practice and standards at this juncture, based on the simply supported boundary conditions. However, in this study, we have proved Two theoretical methods were employed to investigate such situation that the boundary conditions along the junction between the webs and the flanges as built-in and the other boundary conditions along the position of the vertical stiffeners are assumed to be simply supported due to the relatively small stiffness of the stiffeners could be reached. The first method using the minimum potential energy concept to obtain a semi-exact equation from which the interaction equation can be deduced. The second method is performed by using a finite element technique (ANSYS software) to investigate the effects of the geometric parameters on the boundary conditions at the juncture.

Keyword: Plate Girder, Stiffened Web, Shear Strength, Minimum Potential Energy, Ansys

1. Introduction

1.1 Background

Plate girders are used extensively in the world and usually obtained by the assemblage of thin steel plates (built-up sections) hence the size of the flange and web are the discretion of the designer, however, it is necessary to produce an efficient design by providing girders of high strength to weight ratio. Finding an efficient section faces conflicting requirements. The web depth must be large to produce low axial force in flanges moreover to reduce the self-weight the web thickness should be minimum, consequently the web plate becomes more slenderness and tends to buckle at relatively low values of applied shear force therefore buckling must be considered in establishing their dimensions. To overcome this problem, vertical and horizontal stiffeners are used to stiffen webs of the plate girders hence, there are several types of plate girders; unstiffened, transversely stiffened, and transversely and longitudinally stiffened. Many researchers assumed that the web panels are simply supported, while others assumed that the juncture behaved like a clamped support, and others proposed intuitively that the boundary condition lies halfway between the simply supported and clamped conditions. To discuss this confusion, other research studies must be performed to achieve a firm situation for definition the boundary condition at the juncture between the web and flanges in designing plate girder webs.

Given the above topic and taking into account the scope of this paper, studies on the web panel behavior of steel plate girders may be classified into studies on the shear behavior of steel plate girders and the effect of the boundary condition. A brief summary of each of them shall be presented as follows:

1.2 Literature Review

1.2.1 Studies on the Elastic shear buckling strength of steel plate girder

Plated structure which assumed to resist shear only may be analyzed and checked by considering the plates as pure shear panels. Such panels may be decks or walls in topside modules. Then all axial membrane stresses need to be carried by the adjoining framing only which should be analyzed and checked accordingly. Web buckling due to shear is essentially a local buckling phenomenon. Depending upon the geometry, the web plate is capable of carrying additional loads considerably above that at which the web starts to buckle due to post-buckling strength. Basler [1,2] was the first to develop a method for considering the reserve strength of plate girders constructed from hot-rolled plates in their post-buckling region. This reserve strength was caused by the action of the tension field. According to the proposed assumptions, the pure shear continues to act on the web until the shear buckling load reaches. After this stage, the principal compressive stress remains constant [2,3,4]. One may arrive at the renowned concept that states in plate girders reinforced with transverse stiffeners, the tension field action is required to be restrained with the aid of flanges and stiffeners so that the webs can develop their desired post-buckling strength [3]. However, in a series of analytical and experimental studies, Lee and Yoo [5,6] and Lee et al. [7,8], showed that the flanges and transverse stiffeners do not necessarily behave as anchors. In Basler's method, the elastic shear buckling strength of web plates, V_{cr} , subjected to pure shear loading is computed using the classical plate buckling equation, which is given by Timoshenko and Gere [9] as.

$$\tau_{cr} = K \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{d}\right)^2 \quad (1)$$

Where, K is the elastic shear buckling coefficient, which is dependent on the boundary conditions and aspect ratio of the web panel (Φ). Where the elastic shear buckling coefficient, K_{ss} of a web with simply supported conditions at all four edges was originally used in Basler's method [4,5,9] as.

$$K_{ss} = 4 + \frac{5.34}{\phi^2} \quad \text{For } \phi = \frac{a}{d} < 1 \quad (2a)$$

$$K_{ss} = 5.34 + \frac{4}{\phi^2} \quad \text{For } \phi = \frac{a}{d} \geq 1 \quad (2b)$$

Basler assumed that web panels behave similar to simply supported plates since flanges are not stiff enough to provide a desirable torsional rigidity. Based on over 300 numerical results Basler modified this method. According to Lee et al. [10] who investigated the shear behavior of steel I-girders, a new method named Basler's modified method was proposed to calculate the elastic shear buckling coefficient which depends on the ratio of flange thickness to web thickness, $\left(\frac{t_f}{t_w}\right)$. The real boundary condition of the web panel was found to be closer to the clamped case in the range of practical design parameters of plate girders. Besides, to calculate the elastic shear buckling load using Basler's method, they proposed the following relations:

$$K_{sf} = \frac{5.34}{\phi^2} + \frac{5.34}{\phi} + 8.39\phi - 3.44 \quad \text{For } \phi = \frac{a}{d} < 1 \quad (3a)$$

$$K_{sf} = 8.98 + \frac{5.61}{\phi^2} - \frac{1.99}{\phi^3} \quad \text{For } \phi = \frac{a}{d} \geq 1 \quad (3b)$$

$$K_{Lee} = K_{ss} + \frac{4}{5}(K_{sf} - K_{ss}) \left[1 - \frac{2}{3}\left(2 - \frac{t_f}{t_w}\right)\right] \quad \text{for } 0.5 \leq \frac{t_f}{t_w} < 2 \quad (3c)$$

$$K_{Lee} = K_{ss} + \frac{4}{5}(K_{sf} - K_{ss}) \quad \text{for } \frac{t_f}{t_w} \geq 2 \quad (3d)$$

Where K_{ss} and K_{sf} are the elastic shear buckling coefficient of the simple-simple case and simple-fixed case respectively, which signifies the usual case in steel plate girders, may imply that the web-flange junction is 80% clamped.

$$K_{cr} = 0.2K_{ss} + 0.8K_{sf}$$

However, in these equations the flange width is not taken into account, ignoring an important dimension in the surrounding member's rigidity

Al-AZZawi, Z and et (11) have modified the equation for determining the critical buckling shear stress coefficients, the original equation was proposed by Lee et. al [10], but some of the numerically tested specimens in this work were found out of their bounds leading to a non-safe design. This equation was modified using the data analyzed and a new modified equation is presented. To account for the new data in this study will result in the following new equation:

$$K_{AL-Azzawi} = K_{ss} + \frac{4}{5}(K_{sf} - K_{ss}) \left[1 - \frac{2}{3} \left(3 - \frac{t_f}{t_w} \right) \right] \quad \text{for } 1.5 \leq \frac{t_f}{t_w} < 3 \quad (4a)$$

$$K_{AL-Azzawi} = K_{ss} + \frac{4}{5}(K_{sf} - K_{ss}) \quad \text{for } \frac{t_f}{t_w} \geq 3 \quad (4b)$$

Also, Al-Azzawi (11) has studied The effect of stiffener rigidity, which assumed conservatively that the stiffener is stiff only enough to provide simply supported boundary condition at its junction with the web plate, that's found that the true for specimens with aspect ratio, $a_w/h_w > 1.0$, while this is not true for specimens with aspect ratio, $a_w/h_w = 1.0$. An equation taking the stiffener rigidity effect in increasing the buckling coefficient is proposed.

$$k_{stiffener\ Effect} = k + \left(0.5 \ln \left(\frac{t_s}{t_w} \right) \right) \quad (5a)$$

K_s , must be reduced for specimens with a slenderness ratio $\frac{h_w}{t_w} \leq 125$ by the following factor:

$$k_{125\ Effect} = k + \left(\frac{a_w}{h_w} - 1 \right) \quad (5b)$$

Eurocode 3 [13] shear-strength provisions. An adaptation of Höglund's method [14] for non-rigid end posts was proposed by Daley et al. [4], vetted by the AISC Specification Committee, and adopted into the AISC 360–16 [15] specification. According to AISC 360–16 (Chapter G) [15] which is inspired by Basler's theory, the design shear strength of web plates, $V_u = \phi_v V_n$, with relatively small initial geometric imperfection, $D/120000$ [5], shall be determined as.

$$V_n = 0.6 \sigma_{yw} h t_w \left[\frac{1.51 k_{AISC} E}{\left(\frac{h_w}{t_w} \right)^2 \sigma_{yw}} \right] \quad \text{For } \lambda_w > 1.37 \quad (\text{Elastic Buckling}) \quad (6a)$$

$$V_n = 0.6 \sigma_{yw} h t_w \left[\frac{1.1}{\left(\frac{h_w}{t_w} \right)} \sqrt{\frac{k_{AISC} E}{\sigma_{yw}}} \right] \quad \text{For } 1.1 < \lambda_w < 1.37 \quad (\text{Inelastic Buckling}) \quad (6b)$$

$$V_n = 0.6 \sigma_{yw} h t_w \quad \text{For } \lambda_w \leq 1.1 \quad (\text{Plastic Buckling}) \quad (6c)$$

where V_n is nominal shear strength, $\phi_v=0.9$ is the resistance factor for shear, σ_{yw} is the web plate yield stress, h_w is the girder overall depth, and λ_w is the non-dimensional web slenderness parameter determined as.

$$\lambda_w = \frac{h_w}{t_w} \sqrt{\frac{\sigma_{yw}}{k_{AISC} E}} \quad (7)$$

Also, K_{AISC} is the elastic shear buckling coefficient according to AISC specifications written as.

$$K_{AISC} = 5 + \frac{5}{\phi^2} \quad \text{For } \Phi = \frac{a_w}{h_w} < 3 \quad (8a)$$

$$K_{AISC} = 5 \quad \text{For } \Phi = \frac{a_w}{h_w} \geq 3 \quad (8b)$$

2-Theoretical Analysis

2.1-Minimum Potential Energy Method

This method used in this study depends on the assumption of a suitable deflection form (w) that satisfies the assumed end conditions. This deflection function can satisfy the boundary conditions of the present study as built-up edges at the position of the flanges and simply supported edges at the position of the vertical stiffeners as shown in Fig.(1):

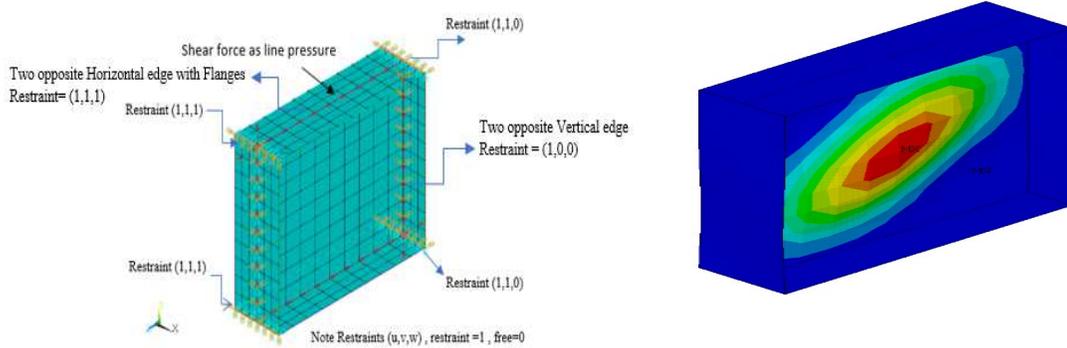


Fig.(1): Boundary condition and loading pattern of models in the FE analysis

Thus, we have,

$$U_B = \frac{D}{2} \int_0^a \int_0^b \left[\left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} \right)^2 - 2(1-\nu) \left(\frac{\partial^2 w}{\partial x^2 \partial y^2} - \left(\frac{\partial^2 w}{\partial x \partial y} \right)^2 \right) \right] dx dy \quad (10)$$

After substituting with w and performing the required integrations, we get;

$$U_B = \frac{D}{2} \left[\left(\frac{3}{16} \frac{\pi^4 b}{a^3} + \frac{\pi^4 a}{b^3} + \frac{1}{2} \frac{\pi^4}{ab} \right) A_1^2 + \left(8 \frac{\pi^4 b}{a^3} + \frac{41}{2} \frac{\pi^4 a}{b^3} + 20 \frac{\pi^4}{ab} \right) A_2^2 - 2(1-\nu) \left(\frac{1}{4} A_1^2 \frac{\pi^4}{ab} + 10 A_2^2 \frac{\pi^4}{ab} - \frac{1}{4} A_1^2 \frac{\pi^4}{ab} + 10 A_2^2 \frac{\pi^4}{ab} \right) \right] \quad (11)$$

In the final form, the strain energy is given by:

$$U_B = \frac{D}{2} [C_1 A_1^2 + C_2 A_2^2] \quad (12)$$

$$\text{Where: } C_1 = \left(\frac{3}{16} \frac{\pi^4 b}{a^3} + \frac{\pi^4 a}{b^3} + \frac{1}{2} \frac{\pi^4}{ab} \right), \quad C_2 = \left(8 \frac{\pi^4 b}{a^3} + \frac{41}{2} \frac{\pi^4 a}{b^3} + 20 \frac{\pi^4}{ab} \right)$$

The work done by the applied forces;

$$T = -\frac{256}{45} \tau_{av} t A_1 A_2 \quad (13)$$

Now, the total potential energy of the system is expressed as:

$$V = U_B + T \quad \text{Thus;}$$

$$V = \frac{D}{2} \left[\left(\frac{3}{16} \frac{\pi^4 b}{a^3} + \frac{\pi^4 a}{b^3} + \frac{1}{2} \frac{\pi^4}{ab} \right) A_1^2 + \left(8 \frac{\pi^4 b}{a^3} + \frac{41}{2} \frac{\pi^4 a}{b^3} + 20 \frac{\pi^4}{ab} \right) A_2^2 - 2(1-\nu) \left(\frac{1}{4} A_1^2 \frac{\pi^4}{ab} + 10 A_2^2 \frac{\pi^4}{ab} - \frac{1}{4} A_1^2 \frac{\pi^4}{ab} + 10 A_2^2 \frac{\pi^4}{ab} \right) \right] - \frac{256}{45} \tau_{av} t A_1 A_2$$

$$V = \frac{D}{2} [C_1 A_1^2 + C_2 A_2^2] - \frac{256}{45} \tau_{av} t A_1 A_2 \quad (14)$$

Minimizing this value requires that: $\frac{\delta V}{\delta A_1} = C_1 D A_1 - \frac{256}{45} \tau_{av} \cdot t \cdot A_2$ and $\frac{\delta V}{\delta A_2} = C_2 D A_2 -$

$\frac{256}{45} \tau_{av} \cdot t \cdot A_1$ Probably the best-known energy technique is the Rayleigh-Ritz procedure

which requires that: $\frac{\delta V}{\delta A_1} = 0, \frac{\delta V}{\delta A_2} = 0$ Thus, we arrive at the following relationship:

$$\tau_{cr} = \frac{3.47D\pi^2}{b^2t} \sqrt{\frac{12+164\phi^2+62\phi^2+242\phi^6+174.75\phi^4}{32\phi^6}} \quad (15)$$

Which is the final required equation which links the critical shear stress in the case of a rectangular plate subjected to shear stress with the prescribed boundary conditions

$$\tau_{cr} = \frac{E\pi^2}{12(1-\nu^2)} \left(\frac{t}{d}\right)^2 \sqrt{\frac{12+164\phi^2+62\phi^2+242\phi^6+174.75\phi^4}{2.657\phi^6}}$$

$$K = \sqrt{\frac{12+164\phi^8+62\phi^2+242\phi^6+174.75\phi^4}{2.657\phi^6}} \quad \text{where } \phi = \frac{a_w}{h_w} \quad (16)$$

3- Comparison with Previous Test Data

To clarify the differences between the shear buckling coefficient resulted in this study and the previous coefficients, Table [1] have been done to indicate the relationships between the proposal coefficient and elastic shear buckling stress the ones belong to other investigators, compared to Lee and Yoo [6] which conducted an experimental study of steel plate girders with non-rigid end posts. In that research, 10 scaled plate girder models were tested to investigate the shear behavior of web panels up to failure. The girders were simply supported with a concentrated load applied at the mid-span. All the geometric and material properties, as well as the support and loading conditions of the aforementioned experiment, are available in Refs. [6]. In this study, to evaluate the proposed simulation technique, the test models are selected and the results obtained from numerical analysis are compared with those of experiments.

Table (1): The elastic shear buckling load (kN) of propped Eq compared to the Ref [6] & Eq. (16).

Model	t/t_w	Coefficient of Buckling (K _q)	Elastic shear buckling loading (KN)
-------	---------	---	-------------------------------------

	$\frac{a_w}{h_w}$	$k_q^{Prop Eq}$	$k_q^{Lee Test}$	k_{sf}^{Basler}	$k_{ss}^{Basler()}$	k_q^{AISC}	$V_{cr}^{prop Eq}$	$V_{cr}^{Lee Test}$	$\frac{V_{cr}^{Prop Eq}}{V_{cr}^{Lee Test}}$	$V_{cr}^{Basler()}$	$\frac{V_{cr}^{Prop Eq}}{V_{cr}^{Basler}}$	$V_{cr}^{Cardiff}$	$\frac{V_{cr}^{Prop Eq}}{V_{cr}^{Cardiff}}$	V_{cr}^{AISC}	
	(Φ)														
G1	1	3.75	15.70	9.34	15.63	9.34	10	288.5	282.43	1.02	278.31	1.03	304.90	0.95	243.5
G2	1	2.5	15.70	9.34	15.63	9.34	10	311.36	332.45	0.94	343.63	0.91	351.73	0.89	243.5
G3	1	3.75	15.70	9.34	15.63	9.34	10	311.36	337.35	0.93	343.63	0.91	386.60	0.81	243.5
G4	1.5	3.75	6.25	6.37	10.88	6.37	7.2	271.57	268.80	1.01	250.68	1.08	292.98	0.93	263.8
G5	1.5	2.5	6.25	6.37	10.88	6.37	7.2	287.49	286.35	1.004	285.47	1.007	284.43	1.01	175.8
G6	1.5	5	6.25	6.37	10.88	6.37	7.2	322.43	312.83	1.03	285.47	1.12	309.26	1.04	175.8
G7	2	2.5	8.86	5.33	10.13	5.33	6.3	250.73	258.90	0.96	231.14	1.08	228.99	1.09	152.2
G8	2	3.75	8.86	5.33	10.13	5.335	6.3	250.73	276.45	0.94	231.14	1.08	251.64	0.99	152.2
G9	3	2.5	5.57	4.60	9.53	4.60	5	164.4	161.81	1.02	209.76	0.78	214.08	0.77	182.6
G10	3	3.75	5.57	4.60	9.53	4.60	5	185.82	194.57	0.96	209.76	0.89	225.06	0.83	182.6

4- Finite Element Modeling

The literature indicates that several studies in shear behavior, based on the slenderness ratio of the web plate parameter, h_w/t_w , have addressed their result, which is unable to account for the effect of web plate boundaries in web shear behavior. [4,12]. Accordingly, conducting a comprehensive study that considers the effects of the flange-to-web stiffness ratio, aspect ratio, and non-dimensional web plate slenderness parameter (λ) on the elastic and plastic shear buckling behavior is necessary. So, several FEM models for a thorough parametric study are developed here. Using these models, the effect of the above-mentioned parameters on the elastic shear buckling strength and shear strength of girders are studied. To investigate the research objectives, 150 models including 16 girders with slender, non-compact, and compact webs, along with non-compact and compact flanges were constructed using the FEM software, ANSYS® platform [16]. Shell element 181 with four nodes, each with six degrees of freedom, and a reduced integration scheme was selected.

As shown in Fig. 2, eight girders with span lengths of $a=1000$ mm, $a=2000$ mm, and $a=700$ mm, equal web depth of $h_w=1000$ mm, and equal flange width of $bf=300$ mm are selected. Considering these two girders, 16 models with various values of web and flange thickness are generated. The details and names of the models are presented in Table 3. Selecting a wide range of thickness and slenderness parameters provides the possibility for studying the elastic/plastic buckling behavior of slender and stocky plates.

4-1 Material properties

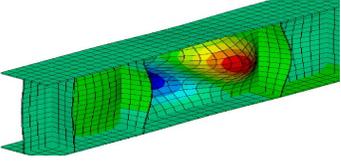
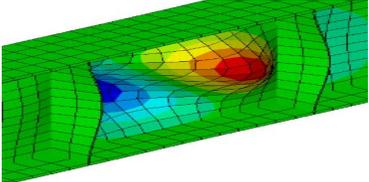
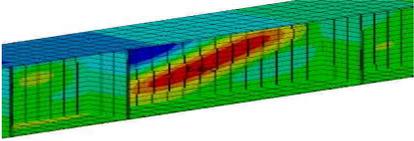
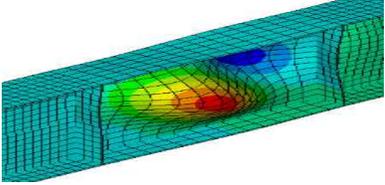
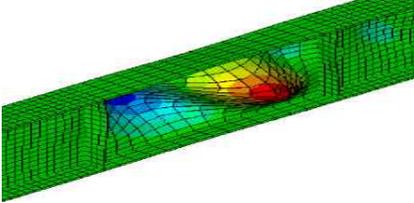
The material behavior of the steel is assumed to be elastic-perfectly plastic. The flange material has Young's modulus E of 200GPa, the normal yield stress of 303.8MPa, and Poisson's ratio of 0.3. Besides, the web plate material has Young's modulus of 200 GPa, the normal yield stress of 318.5MPa, and Poisson's ratio of 0.3. It must be noted that these material properties are selected from G5 test model in Ref. [6].

4-2 Verification of the numerical modeling process

Lee and Yoo [6] conducted an experimental study of steel plate girders with non-rigid end posts. In that research, 10 scaled plate girder models were tested to investigate the shear behavior of web panels up to failure. The girders were simply supported with a concentrated load applied at the mid-span. All the geometric and material properties, as well as the support and loading conditions of the aforementioned experiment, are available in Refs. [6,17]. In this study, to evaluate the proposed simulation technique, the test model G2 is selected and the results obtained from numerical analysis are compared with those of experiments. According to the test report, the total shear strength of G2 girder is equal to 309.36kN, which is very close to $V_u = 332.45$ kN as the ultimate shear strength predicted

by the FEM simulation, with a maximum difference of -1.38% . As seen in Fig. 2, there is a good agreement between the FEM and that of the experimental test both in terms of failure mode and overall behavior.

Table (2): Comparison between the presented FE model, theoretical analysis, and the tested model presented by Lee (6).

Plate model	The presented FE model	V_{FE} (KN)	The tested model presented by Lee et al. [6]	$V_{Lee,et.al}$ (KN)	$\frac{V_{FE}}{V_{Lee,Test}}$
	The elastic buckling shapes		The failure tested shape		
G2		339.38		332.45	1.02
G3		311.07		337.35	0.92
G5		263.4		286.35	0.92
G7		233.66		258.90	0.90
G8		302.20		276.45	1.09

4-3 Parametric Study

To investigate the effect of the different properties of the cross-section on calculating the critical load of the plate girder a large number of numerical analyses have been carried out on several combinations of three key geometric variables, $\frac{a_w}{h_w}$, $\frac{h_w}{t_w}$ and $\frac{t_f}{t_w}$. The ranges of parameters examined in this study are listed in Table (3):

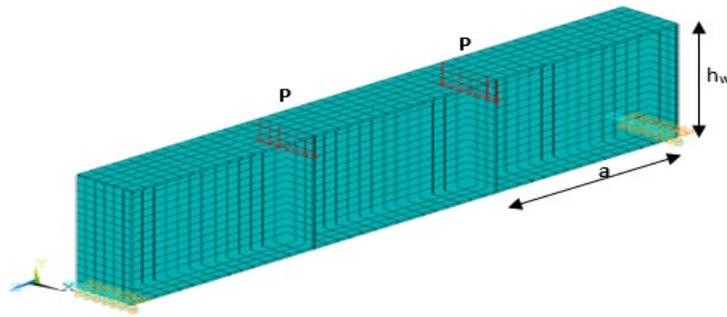


Fig. 4. FE model of flat web under shear and edge restraints.

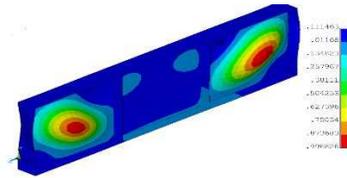
Table:(3): Classification of Plate girder Models

Model	a_w	h_w (mm)	t_w (mm)	b_f (mm)	t_f (mm)	h_w/t_w	$\phi = a_w/h_w$	t_f/t_w
PLG1					10			2.5
PLG2	1000	1000	4		15	250	1	3.75
PLG3					20			5
PLG4				200	10			2.5
PLG5					10			2
PLG6	1000	1000	5		15	200	1	3
PLG7					20			4
PLG8					10			2
PLG9			6		10	166.67	2	1.67

PLG10	2000	1000		15			2.5
PLG11				20			3.33
PLG12				10			1.67
PLG13				20			2.5
PLG14				15			1.87
PLG15	700	1000	8	20	125	0.70	2.5
PLG16				15			1.87

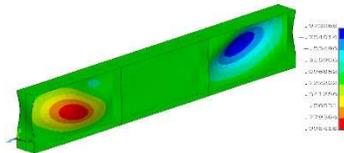
PLG1 – 1st Mode

$V_n=103.43\text{KN}$



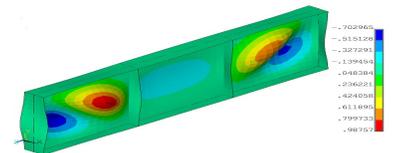
PLG1 – 2nd Mode

$V_n=103.63\text{KN}$



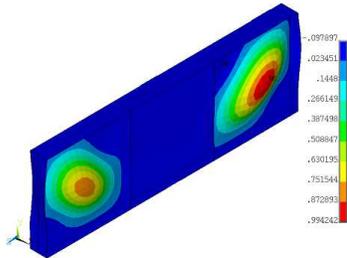
PLG1 – 3rd Mode

$V_n=113.65\text{KN}$



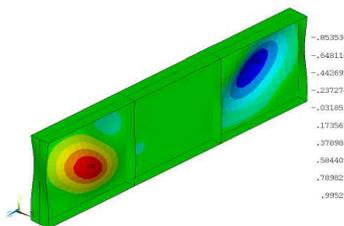
PLG7 – 1st Mode

$V_n=205.07\text{KN}$



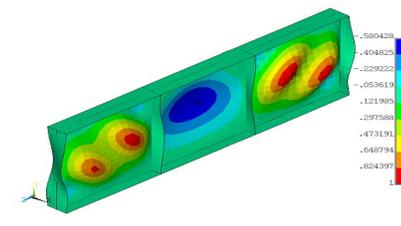
PLG7 – 2nd Mode

$V_n=205.45\text{KN}$



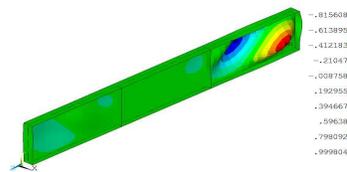
PLG7 – 3rd Mode

$V_n=411.2\text{KN}$



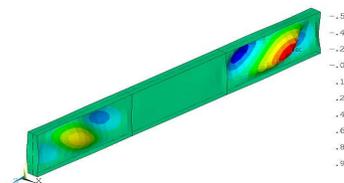
PLG11 – 1st Mode

$V_n=314.7\text{KN}$



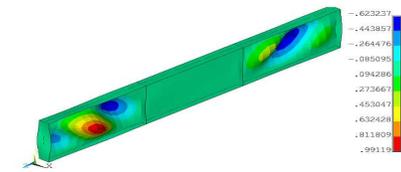
PLG11 – 2nd Mode

$V_n=320.6\text{KN}$



PLG11 – 3rd Mode

$V_n=326.99\text{KN}$

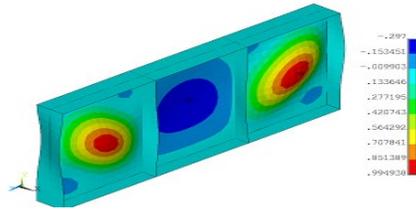


PLG14 – 1st Mode

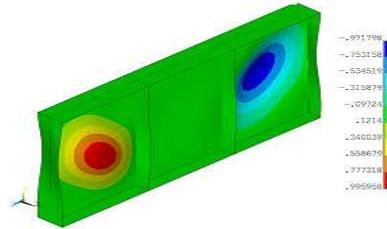
PLG14 – 2nd Mode

PLG14 – 3rd Mode

$V_n=1026.47\text{KN}$



$V_n=1045.6\text{KN}$



$V_n=1140.45\text{KN}$

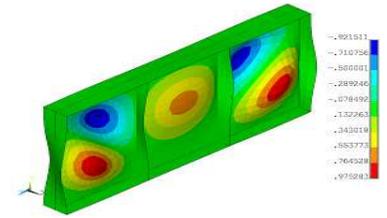


Fig.(5): PG1, PG7, PL11, and PG14 eigen-buckling mode shapes

5- Results and Discussion

The study presented in this paper which is, mainly, based on an accurate theoretical procedure supported by a finite element check for the behavior of the plate girder under pure shear, can be used to obtain a variety of results. The careful inspection of these results gives the following conclusions:

A comprehensive theoretical analysis is performed to obtain a closed-form solution for the critical shear stress of the web of the plate girder. The boundary conditions for the plate panel were, also chosen to satisfy the real situation.

- 1- For aspect ratio equal 1.0 the effect of flange thickness to web thickness on critical load is obvious. When tf increases, the boundary condition at web- flange juncture is closer to fixed end conditions.
- 2- It is also clear for aspect ratio equal 1.0 that whenever the slenderness ratio decreases, the boundary condition at web- flange juncture loses its fixation condition and acts as a simple end condition.

- 3- For aspect ratio equal 0.7 the effect of flange thickness to web thickness ratio has no effective influence on the boundary condition, as the shear load has the same value at all flange-to-web thickness ratio especially for slenderness ratio more than 200
- 4- For aspect ratio equal to 0.7 it was concluded that whenever the thickness of web increases, the boundary condition at web- flange juncture loses its fixation condition.
- 5- In the case of low and high slenderness ratio plate with aspect ratio equal to 1.0, it is clear that the critical shear load produced by energy method considering simple condition at the juncture between web and flange is less than the critical shear load produced by other methods discussed before.
- 6- In the case of a low slenderness ratio plate with an aspect ratio equal to 0.7, it is clear that the critical shear load produced by the energy method considering either simple or clamped condition at the juncture between web and flange is around the critical shear load produced by other methods discussed before.

6- Conclusion

The theoretical analysis and finite element analysis for predicting the behavior of plates and stiffened panels under pure shear which has been presented in this study was used to obtain a variety of results. These resulting data lead to the following conclusions:

1- The parametric study performed by the finite element technique showed that it is more realistic to consider the boundary conditions between the web and the flanges as built-in edges due to the high rigidity of the flanges compared to that of the vertical stiffeners.

2- By using the principle of minimum potential energy, it was possible to obtain an expression from which the critical shear stress for simple-built –in plate panels can be obtained.

The results from this equation were compared with the results from the Finite element analysis where a satisfactory agreement was noticed.

3- Applying the obtained results by calculating the critical and allowable shear stresses for panels under pure shear, showed that a more economical, yet safe, design can be obtained for the web of such panels.

Declarations

Conflict of interest on behalf of all authors, the corresponding author states that there is no conflict of interest.

References

[1]-K. Basler, J.A. Mueller, B. Thurlimann, B.T. Yen, K. Basler, J. Mueller, Web buckling tests on welded plate girders, Overall Introduction, and Part 1: The Test Girders. WRC Bulletin, 64, (September 1960), Reprint No. 165 (60-5), WRC Bull., 64, 1960, pp. 60–65.

[2] K. Basler, Strength of plate girders in shear, Proc. ASCE, 87, (ST7), (October 1961), Reprint No. 186 (61–13), Fritz Lab. Reports, 1961

- [3] C.H. Yoo, S.C. Lee, Mechanics of web panel post-buckling behavior in shear, J. Struct. Eng. 132 (2006) 1580–1589, [https://doi.org/10.1061/\(ASCE\)0733-9445\(2006\)132:10\(1580](https://doi.org/10.1061/(ASCE)0733-9445(2006)132:10(1580).
- [4] A.J. Daley, D. Brad Davis, D.W. White, Shear strength of unstiffened steel I-section members, J. Struct. Eng. 143 (2017), 04016190. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001639](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001639).
- [5] S.C. Lee, C.H. Yoo, Strength of plate girder web panels under pure shear, J. Struct. Eng. 124 (1998) 184–194, [https://doi.org/10.1061/\(ASCE\)0733-9445\(1998\)124:2\(184](https://doi.org/10.1061/(ASCE)0733-9445(1998)124:2(184).
- [6] S.C. Lee, C.H. Yoo, Experimental study on ultimate shear strength of web panels, J. Struct. Eng. 125 (1999) 838–846, [https://doi.org/10.1061/\(ASCE\)0733-9445\(1999\)125:8\(838](https://doi.org/10.1061/(ASCE)0733-9445(1999)125:8(838).
- [7] S.C. Lee, C.H. Yoo, D.Y. Yoon, Behavior of intermediate transverse stiffeners attached on web panels, J. Struct. Eng. 128 (2002) 337–345.
- [8] S.C. Lee, C.H. Yoo, D.Y. Yoon, New design rule for intermediate transverse stiffeners attached on web panels, J. Struct. Eng. 129 (2003) 1607–1614.
- [9] S.P. Timoshenko, J.M. Gere, Theory of Elastic Stability, Courier, Dover Publications, 2009.
- [9] M.E.M. Garlock, J.D. Glassman, Elevated temperature evaluation of an existing steel web shear buckling analytical model, J. Constr. Steel Res. 101 (2014) 395–406, <https://doi.org/10.1016/J.JCSR.2014.05.021>.
- [10] S.C. Lee, J.S. Davidson, C.H. Yoo, Shear buckling coefficients of plate girder web panels, Comput. Struct. 59 (1996) 789–795, [https://doi.org/10.1016/0045-7949\(95\)00325-8](https://doi.org/10.1016/0045-7949(95)00325-8).

[11] Z. Al-Azzawi, T.Stratford, M. Rotter, L. Bisby, Effect of flange and stiffener rigidity on the boundary conditions and shear buckling stress of plate girders, Conference: 16th European Bridge Conference (2015). <https://www.researchgate.net/publication/303803332>.

[12] S.C. Lee, D.S. Lee, C.H. Yoo, Ultimate shear strength of long web panels, J. Constr. Steel Res. 64 (2008) 1357–1365, <https://doi.org/10.1016/J.JCSR.2008.01.023>

[13] EN 1993-1-5, Eurocode 3: Design of Steel Structures - Part 1–5: Plated Structural Elements, European Committee for Standardization (CEN), 2006.

[14] T. Höglund, Shear buckling resistance of steel and aluminum plate girders, Thin-Walled Struct. 29 (1997) 13–30, [https://doi.org/10.1016/S0263-8231\(97\)00012-8](https://doi.org/10.1016/S0263-8231(97)00012-8).

[15] ANSI/AISC 360-16, Specification for Structural Steel Buildings, American Institute of Steel Construction, 2016.

[16] ANSYS®, Release 15.0, ANSYS, Inc., 2007.

[17] A. Reis, N. Lopes, E. Real, P.V. Real, Numerical modeling of steel plate girders at normal and elevated temperatures, Fire Safe. J. 86 (2016) 1–15, <https://doi.org/10.1016/j.firesaf.2016.08.005>.