ASGE Vol. 06 (01), pp. 68-77, 2022

Egyptian Knowledge Bank

International Journal of Advances in Structural and Geotechnical Engineering

https://asge.journals.ekb.eg/

Print ISSN 2785-9509 *Online ISSN 2812-5142*

A New Approach to Estimate the Shear Strength of Stiffened Plate Girder

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ABSTRACT

Although critical bucking shear stress of plate girders loaded mainly in shear have been studied for so many years by multiple researchers, but the matter of choosing the right boundary conditions at the junction between the web plate and the flange & stiffener is still a problem of Engineering judgment and standards approach. all current codes of practice and standards adopt a conservative design criterion by taking the simply supported boundary conditions at this junction. 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.

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

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.

$$K_{ss} = 4 + \frac{5.34}{\phi^2} \qquad For\ \ \phi = \frac{a}{d} < 1 \tag{2a}$$

$$K_{ss} = 5.34 + \frac{4}{\phi^2} \qquad For\ \ \phi = \frac{a}{d} \geq 1 \tag{2b}$$

Basler[1,2] assumed that web panels behave similarly 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, ($\frac{t_f}{t_w}$). 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:

$$K_{sf} = \frac{5.34}{\emptyset^2} + \frac{5.34}{\emptyset} + 8.39\emptyset - 3.44 \qquad For\ \ \phi = \frac{a}{d} < 1 \tag{3a}$$

$$K_{sf} = 8.98 + \frac{5.61}{\emptyset^2} - \frac{1.99}{\emptyset^3} \qquad For\ \ \phi = \frac{a}{d} \geq 1 \tag{3b}$$

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

$$K_{Lee} = K_{ss} + \frac{4}{5}\left(K_{sf} - K_{ss}\right) \qquad for\ \frac{t_f}{t_w} \geq 2 \tag{3d}$$

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:

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

$$K_{Al-Azzawi} = K_{ss} + \frac{4}{5}\left(K_{sf} - K_{ss}\right) \qquad for\ \frac{t_f}{t_w} \ge 3 \qquad (4b)$$

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.

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

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

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

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.

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

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

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

where V_n is nominal shear strength, ϕ_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.

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

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

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

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

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):

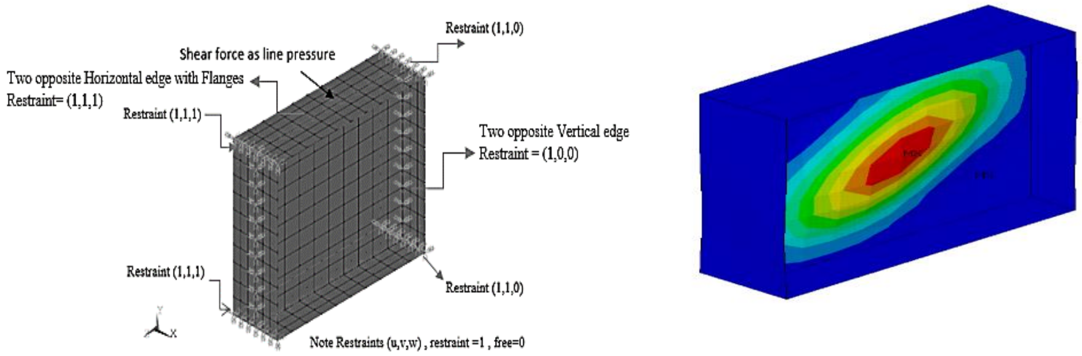


Fig.(1): Boundary condition and loading pattern of models in the FE analysis

Thus, we have,

$$w = A_1 sin\frac{\pi x}{a} sin^2\frac{\pi y}{b} + A_2\, sin\frac{2\pi x}{x}\left(cos\frac{\pi y}{b} - cos\frac{3\pi y}{b}\right) \quad (9)$$

Where A_1 and A_2 are unknown coefficients, The strain energy in bending is given by:

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

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

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

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

$$U_B = \frac{D}{2}[C_1A_1^2 + C_2A_2^2] \quad (12)$$

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

The work done by the applied forces;

$$T = -\frac{256}{45}\tau_{av}tA_1A_2 \quad (13)$$

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

V=U_B+T Thus;

$$V=\frac{D}{2}\left[\left(\frac{3}{16}\frac{\pi^4 b}{a^3}+\frac{\pi^4 a}{b^3}+\frac{1}{2}\frac{\pi^4}{ab}\right)A_1^2 + \left(8\frac{\pi^4 b}{a^3}+\frac{41}{2}\frac{\pi^4 a}{b^3}+20\frac{\pi^4}{ab}\right)A_2^2 - 2(1-\upsilon)\left(\frac{1}{4}A_1^2\frac{\pi^4}{ab}+10A_2^2\frac{\pi^4}{ab}-\frac{1}{4}A_1^2\frac{\pi^4}{ab}+10A_2^2\frac{\pi^4}{ab}\right)\right] - \frac{256}{45}\tau_{av}tA_1A_2$$

$$V=\frac{D}{2}[C_1A_1^2 + C_2A_2^2] - \frac{256}{45}\tau_{av}tA_1A_2 \quad (14)$$

Minimizing this value requires that: $\frac{\delta V}{\delta A_1} = C_1DA_1 - \frac{256}{45}\tau_{av}.t.A_2$ and $\frac{\delta V}{\delta A_2} = C_2DA_2 - \frac{256}{45}\tau_{av}.t.A_1$

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:

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

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

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

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

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	$\frac{a_w}{h_w}$ (Φ)	t_f/t_w	Coefficient of Buckling (Kq)					Elastic shear buckling loading (KN)								
			$k_q^{prop\ Eq}$	$k_q^{Lee\ Test}$	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}
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 (hw/tw), 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=2000mm, and a= 700 mm, equal web depth of h_w =1000 mm, and equal flange width of bf=200mm 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.

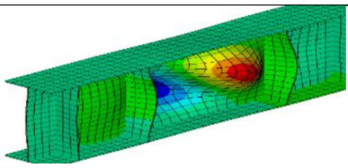
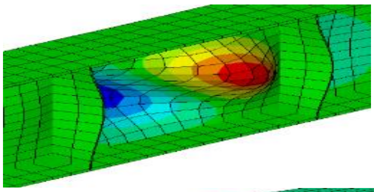
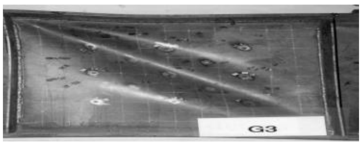
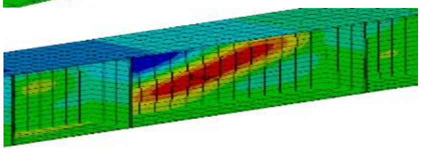
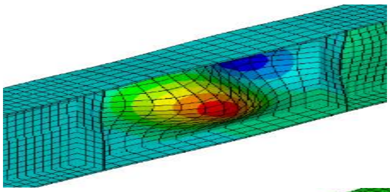
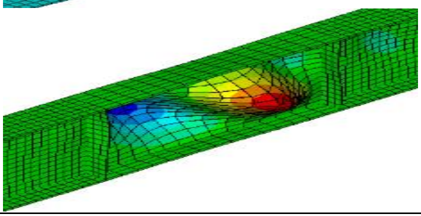
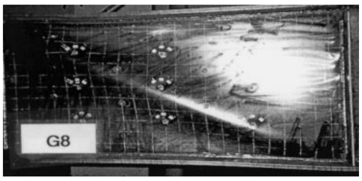
4-1 Material properties

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 G2 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. [6,17,18]. 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 339.38kN, which is very close to Vu = 332.45 kN as the ultimate shear strength predicted by the FEM simulation, with a maximum difference of −1.02%. As seen in Table (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		The tested model presented by Lee et al. [6]		$\frac{V_{,FE}}{V_{Lee,Tes}}$
	The elastic buckling shapes	V_{FE} (KN)	The failure tested shape	$V_{Lee,et.al}$ (KN)	
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

4-۳ 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):

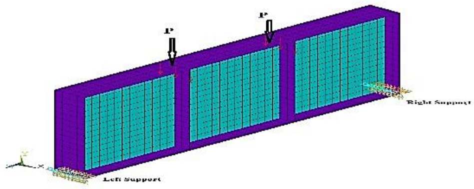


Fig. (2): FE model of flat web under shear and edge restraints.

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	$\emptyset = a_w/h_w$	t_f/t_w
PLG1	1000	1000	4	200	10	250	1	2.5
PLG2					15			3.75
PLG3					20			5
PLG4					25			2.5
PLG5	1000	1000	5		10	200	1	2
PLG6					15			3
PLG7					20			4
PLG8					25			2
PLG9	2000	1000	6		10	166.67	2	1.67
PLG10					15			2.5
PLG11					20			3.33
PLG12					25			1.67
PLG13	700	1000	3		10	125	0.70	2.5
PLG14					15			1.87
PLG15					20			2.5
PLG16					25			1.87

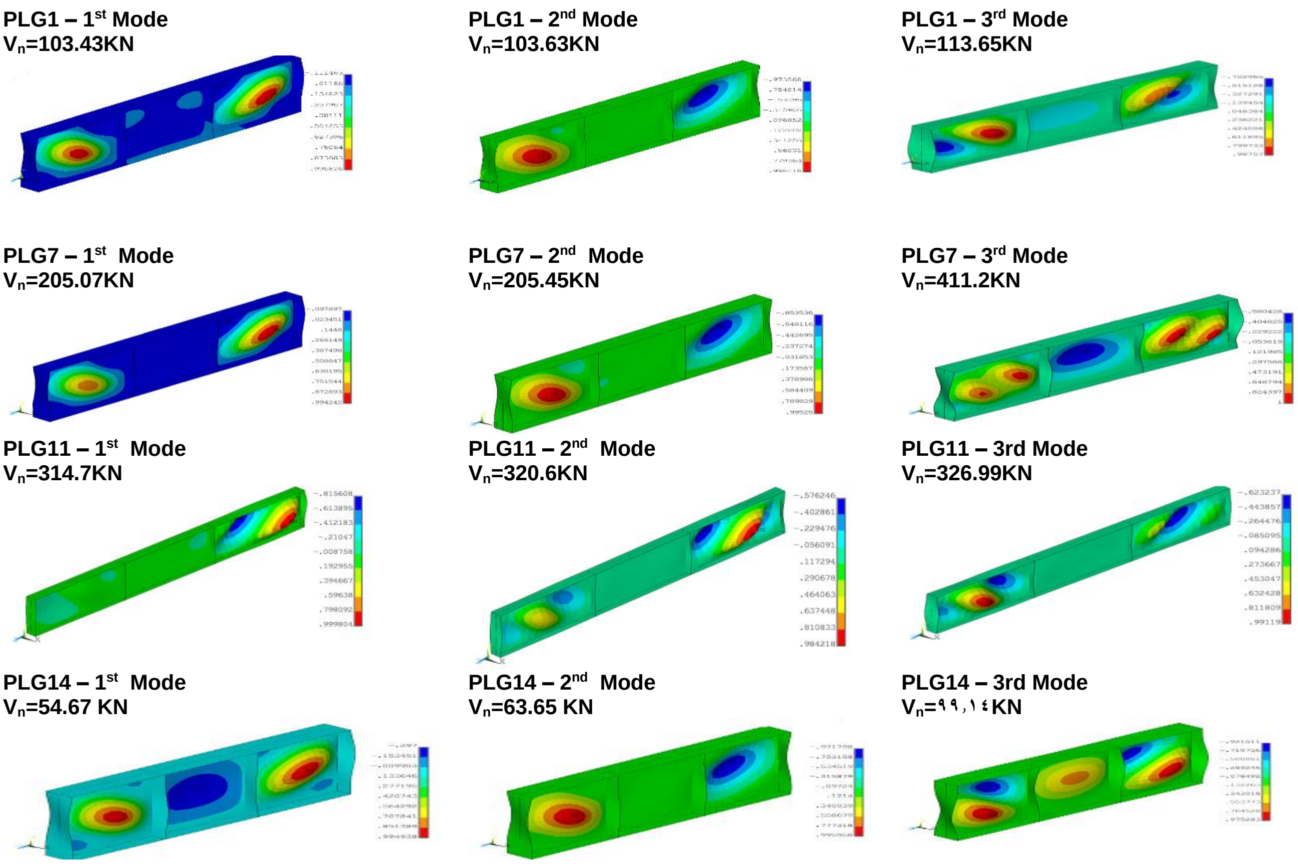


Fig.(3): PG1, PG7, PL11, and PG14 Eigen-buckling mode shapes

Table. (4) Results of the finite element analysis (FEA) for ultimate shear loads.

Model	h_w/t_w	$\emptyset = a_w/h_w$	t_f/t_w	Prop Eq (16) $V_{n\,Prp\,Eq}$ KN	FE for shear Loads (V_U) KN	$V_{U,FE}/V_{n,Prop,Eq(16)}$
PLG1	250	1	2.5	181.44	205.91	1.13
PLG2			3.75	181.44	186.82	1.03
PLG3			5	181.44	183.02	1.008
PLG4			6.25	181.44	184.61	1.02
PLG5	200	1	2	354.35	321.97	0.91
PLG6			3	354.35	378.12	1.06
PLG7			4	354.35	401.97	1.13
PLG8			5	354.35	401.97	1.13
PLG9	166.67	2	1.67	735.61	412.41	0.56
PLG10			2.5	735.61	444.75	0.60
PLG11			3.33	735.61	529.79	0.72
PLG12			4.16	735.61	602.68	0.82
PLG13	333.33	0.7	1.25	96.39	99.12	1.02
PLG14			1.87	96.39	99.14	1.03
PLG15			2.5	96.39	98.79	1.02
PLG16			3.12	96.39	98.56	1.02

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.

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