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Parametric Study of the Shear Web Buckling Behaviour for Steel Girders with Corrugated Webs

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ABSTRACT

Because of its numerous advantages, the steel plate girder with the corrugated web is widely employed in a variety of applications. It has great resistance to shear buckling when compared to another flat web girder of the same steel weight. Some of the present standards and specifications such as the Euro-code include design approaches for determining the shear buckling strength of the corrugated web girders. This research examines the resistance of corrugated web girders to shear buckling, investigates their behavior, and checks the accuracy of the Euro-code equations. ANSYS is used to do the nonlinear finite element analysis. This study considers the effects of several parameters (web thickness, web height, corrugation angle, corrugation breadth, and corrugation shape). It is found that the shear buckling resistance for girders with corrugated webs is increased due to the increase in web thickness, web height, corrugation angle, corrugation depth, and the number of corrugation waves along the girder span while it decreased due to the increased of the corrugation width. The main conclusion is the accuracy of using Euro-code equations for trapezoidal corrugated webs is good but it is must be modified in the case of girders with triangular or rectangular corrugated webs.

1. Introduction

The steel plate girder with the corrugated web has been used on a large scale since the automation welding technologies are available in the late 1980s/early 1990s. It has many advantages such as a high ability to carry loads compared to the weight of the steel used. The previous researches state that the built-up steel sections with the corrugated web have been used particularly to increase the out-of-plane stiffness and shear buckling strength without the use of vertical stiffeners.

Smith [1] performed four tests on two girders with corrugated webs welded to the flanges by using intermittent welding. He found that the connection between the flanges and the web of girder is critical against the shear stresses, where the weld used in the test was subjected to high stresses and the web was easily ruptured at this point before it reached its buckling strength. *Hamilton [2]* performed 42 tests on 21 beams with four different corrugation configurations and two thicknesses of webs. He found that the dense corrugation profiles are more likely to fail due to global shear buckling. *Luo, R. and Edlund, B. [3]* used non-linear finite element analysis to perform a geometrical parametric study. The numerical results were compared with existing empirical and analytical formulae. They found that the ultimate shear capacity increases proportionally with the girder depth and seems not to be dependent on the girder length depth ratio. They also found that the corrugation depth did not affect the ultimate shear

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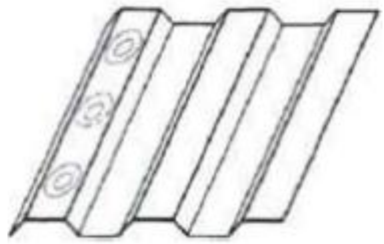
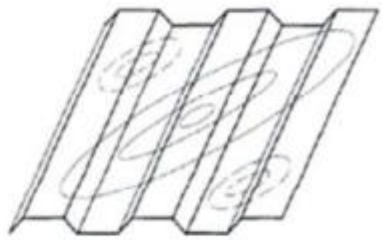
capacity, but the degree of the localization of the buckling mode was affected. *Elgaaly, Hamilton, and Seshadri [4]* tested beams with corrugated webs under shear to study the failure due to the buckling of the web. The test specimens were modeled by using the finite element program ABAQUS. They found that the buckling of the web is local for the coarse corrugations and global for the dense corrugations. *Johnson, R.P and Cafolla [5]* studied the effect of the vertically corrugated webs on the local buckling of the compressive flange and the flexural behavior of plate girders with corrugated webs numerically and experimentally. They found that the contribution of the web to flexural capacity was found to be small. The slenderness should be based on the distances from the horizontal fold to the edges of the flange. *E.Y. Sayed – Ahmed [6]* performed analytical models using ANSYS vr.10 to study the buckling behavior of plate girders with the corrugated web. He found that the behavior of the corrugated webs subjected to pure shear stresses is dependent on the shear buckling and steel yield strength. And also, the webs of these types of girders are affected by two modes of buckling: local buckling and global buckling. An interaction between the two buckling modes represents another possibility of failure. *Sachin K.G et. al. [7]* performed an analytical study by using SAP2000 software to study the buckling performance of plate girders with flat and corrugated webs. The corrugated webs were considered rectangular and triangular shapes. From buckling analysis, they found that the buckling strength of plate girders with corrugated triangular webs increases as a corrugation angle increases. While, in cases of plate girders with corrugated rectangular webs, it increases due to the decrease of corrugation width. *Izni Syahrizal Ibrahim et. al. [8]* performed an experimental and analytical study using LUSAS to develop a suitable equation to calculate critical shear capacity for corrugated web girders with different types of web configurations. They found that the analytical results of critical shear capacity are in good agreement compared with experimental results.

2. Modes of Failure for Corrugated Steel web under shear stress

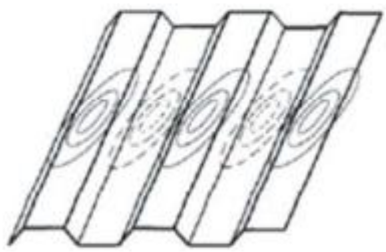
There are three different shear buckling modes of failure (local; global; and interactive), as shown in Fig. (1) that are possible, depending on the geometric characteristics and dimensions of the corrugated steel web *[9]*.



(a) Local buckling mode



(b) Global shear buckling mode



(c) Interactive shear buckling mode

Figure (1) Modes of Failure for Plate Girder with Corrugated Steel web *[9]*

2-1 Local Buckling Mode:

Local buckling occurs when the width to thickness ratio of a flat sub-plate between vertical edges is significant, as seen in Fig. (1-a). In this mode of failure, the corrugated web acts as a series of flat plate sub-panels that mutually support each other along with their vertical (longer) sides and are supported by the flanges at their horizontal (shorter) edges, similar to the instability of a steel panel merely supported between two folds. *Galambos [10]* calculated the elastic buckling stress for these flat plate sub-panels by considering them to be isotropic plates.

2-2 Global shear Buckling Mode:

Global shear buckling is the most common failure mode when the web of the beam has dense corrugations. As seen in Fig. (1-b), global shear buckling is characterized by the production of diagonal buckles across the entire web, comparable to a flat plate web. It is distinguished by diagonal buckling over a number of corrugation panels. This is a common failure mode for dense corrugation *[9]*. The orthotropic-plate buckling theory can be used to calculate the buckling stress when global buckling occurs *[10]*.

2-3 Interactive shear Buckling Mode:

The interactive shear buckling mode influences the shear buckling strength by interacting between local and global shear buckling modes as demonstrated in Fig. (1-c). The critical stress due to the interaction between local and global buckling modes may be calculated using Equ. (1), which is based on experimental evaluations performed by *Bergfelt and Leiva-Aravena [11]*.

$$\frac{1}{\tau_{cr,i}} = \frac{1}{\tau_{cr,l}} + \frac{1}{\tau_{cr,g}} \qquad \text{.........Equ. (1)}$$

Where:

$\tau_{cr,l}$: is the critical shear stress due to local web buckling.

$\tau_{cr,g}$: is the critical shear stress due to global shear buckling.

$\tau_{cr,i}$: is the interactive shear stress buckling.

3. Design code equations

The *Euro-code* [12] gave the design procedure to determine the shear buckling resistance ($V_{(EC3)}$) for steel girders with the corrugated web as shown in Equ. (2).

$$V_{(EC3)} = \chi_c \frac{f_{yw}}{\gamma_{M1}\sqrt{3}} h_w t_w \qquad \text{........Equ. (2)}$$

$$\chi_c = min.\begin{cases} \chi_{c,l} = \left[\frac{1.15}{0.9+\lambda_{c,l}}\right] \leq 1 \\ \chi_{c,g} = \left[\frac{1.5}{0.5+\lambda_{c,g}^2}\right] \leq 1 \end{cases} \qquad \text{......Equ. (3)}$$

Where χ_c is the reduction factor for the contribution of the corrugated web to the shear buckling resistance which is the minimum of local buckling factor $\chi_{c,l}$ and global buckling factor $\chi_{c,g}$ as detailed in Equ. (3).

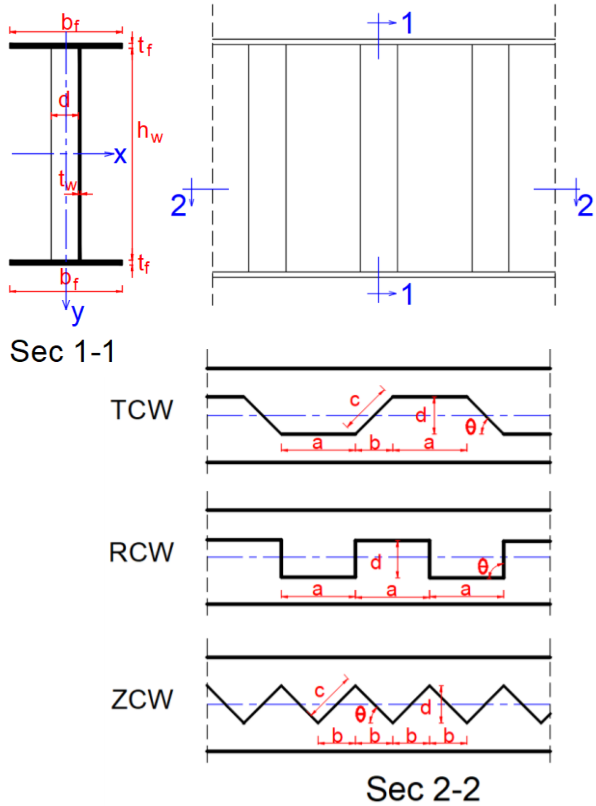


Fig. (2) Different studied web corrugation profiles and the used symbols of different dimensions.

While:

f_{yw} : is the web yield strength.

γ_{M1} : is the partial factor for resistance of members to instability assessed by member checks.

h_w, t_w: are web height and thickness respectively as shown in Fig. (2).

$\lambda_{c,l}$: is the reference slenderness for local web buckling: $\lambda_{c,l}=\sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr,l}}}$

$\tau_{cr,l}$: is the critical shear stress due to local web buckling: $\tau_{cr,l}=4.83E(\frac{t_w}{a_{max}})^2$

a_{max}: is the maximum width of the inclined panel, c, or the horizontal panel, a, of the web.

$$a_{max} = max.\begin{cases} c \\ a \end{cases}$$

$\lambda_{c,g}$: is the reference slenderness for global web buckling: $\lambda_{c,g}=\sqrt{\frac{f_y}{\sqrt{3}\,\tau_{cr,g}}}$

$\tau_{cr,g}$: is the critical shear stress due to global web buckling: $\tau_{cr,g}=\frac{32.4}{h_w^2 t_w}\sqrt[4]{D_y^3 D_x}$

$$D_x = \frac{E t_w^3}{12(1-\nu^2)} \left[\frac{a+b}{a+c}\right]$$

$$D_y = \frac{E t_w d^2}{12} \left[\frac{3a+c}{a+b}\right]$$

ν: is Poisson's ratio,

a, b, c: are the lengths of the horizontal panel, horizontal projection of the inclined panel, and inclined panel respectively of the corrugation web as shown in Figure (2).

d: is the depth of corrugation as shown in Figure (2).

The aims of this paper are:

1) Studying the influence of the variations in the dimensions and corrugation shape of the steel girders with corrugated web on web buckling behavior.
2) Check the accuracy of using the Euro-code equations for calculations of the critical shear buckling resistance by comparing its results with the results of finite element models.

4. Modeling of nonlinear Finite elements

Advanced nonlinear computations were carried out using the commercial finite element package ANSYS [13]. The finite nonlinear Shell element 281 is used to model all of the girders' flanges and webs because it has eight nodes and each node has six degrees of freedom.

4-1 Properties of material

The properties of steel material used in all evaluated girders follow the data of the bilinear stress-strain curve as shown in Fig. (3).

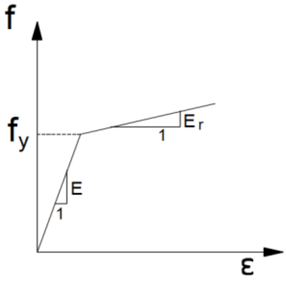


Fig. (3) Bilinear stress-strain curve for steel

It is assumed that the material started with an initial elastic modulus of elasticity is 210 GPa up to specified yield stress f_y equals 240 MPa. After that, it assumed that the material followed the linear hardening with a reduced hardening modulus $\boldsymbol{E_r}$ equal to *0.01E* up to ultimate strength $\boldsymbol{f_u}$ equals *370 MPa*. The Poisson's ratio is assumed to equal 0.3.

4-2 Numerical model and boundary conditions

Similarly, to the boundary condition which was considered in the research of *Jiho Moon et. al. [16]*, the studied girders are considered simply supported in flexure and torsion. Point (A) is considered as a hinge joint, where the translations in X, Y, and Z directions are restrained. While point (B) is considered as a roller joint, where the translations are restrained only in Y, and Z directions. All boundary conditions for analyzed girders are shown in Fig. (4). To study the web buckling behavior for all analyzed girders under pure shear, an additional boundary condition was added by supporting the upper and lower flanges for all models against lateral torsional buckling by restraining displacement in Z-direction as shown in Fig. (5).

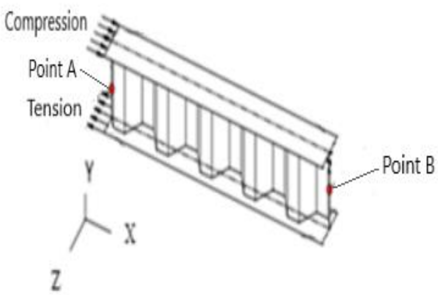


Fig. (4) Boundary conditions of the analyzed girders

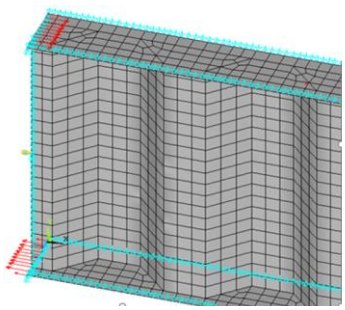


Fig. (5) additional boundary condition was added to FEM

The critical shear buckling stress of thin plates was at first studied by Timoshenko al. et. [14]. They got analytical expressions for obtaining the critical shear buckling stress of thin plates with boundary conditions of simply supported edges as shown in Fig. (6). Therefore, to make all analyzed girders with pure shear stresses without being affected otherwise, the loading condition assumed by Timoshenko was taken into account by loading all the beams with tension and compression forces at the ends of the top and lower flanges from one side shown in Figs. (4) and (5). Eigenvalue buckling analysis was performed on all analyzed girders to evaluate the theoretical buckling loads where girders become unstable.

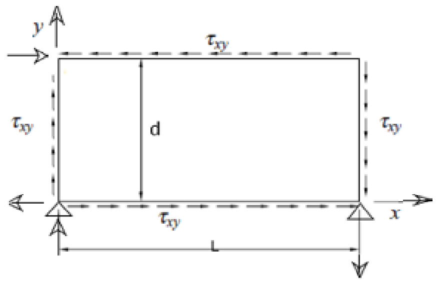


Fig. (6) Indication of web plate with conditions of edges simply supported and subjected to a state of shear stress.

4-3 Geometry of the analyzed girders

The analyzed girders are assumed simply supported with span (L=15600mm), consisting of two equal flange plates and corrugated web with different profiles and dimensions as shown in Fig. (2) and detailed in Table (2). The width and thickness of flanges for all models are equal to 500mm and 40mm, respectively. The web is considered as a corrugated web with height and thickness equal to 2000mm and 12 mm, respectively. The corrugation angle is assumed to be equal 45º. The width (b) and depth of corrugation (d) are taken equal 330mm and 270mm, respectively, as shown in Fig. (7).

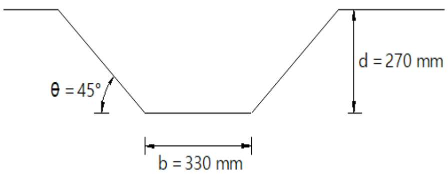


Fig. (7) Web corrugation profile

5. Verification of the finite element model

The experimental model was performed by *Nikolaus L. [15]* to investigate the maximum patch load of a (CW) girder at different locations. The girder consists of two equal spans (3.0m) with hinge support at point (a) and roller supports at points (b) and (c). Vertical stiffeners were arranged at each support. The web thickness and height of the analyzed girder are 3mm and 578mm respectively. Both flanges are made from 160mm wide and 12mm thick steel plates. The corrugation angle equals 45º and the width (b) and the depth (h) are equal to 140mm and 50mm, respectively, as shown in Figs. (8) and (9).

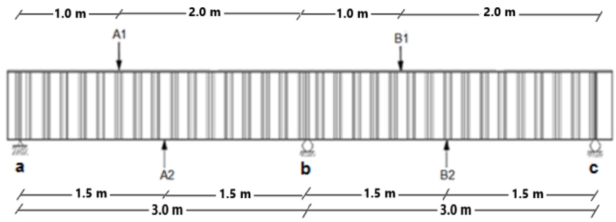


Fig. (8) An illustration showing the tested girder and the load cases by *Nikolaus L. [15]*

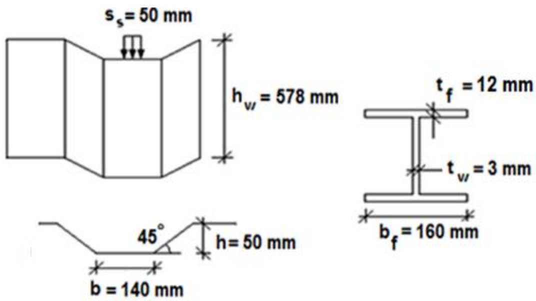


Fig. (9) Dimensions and geometry of the tested girder by *Nikolaus L. [15]*.

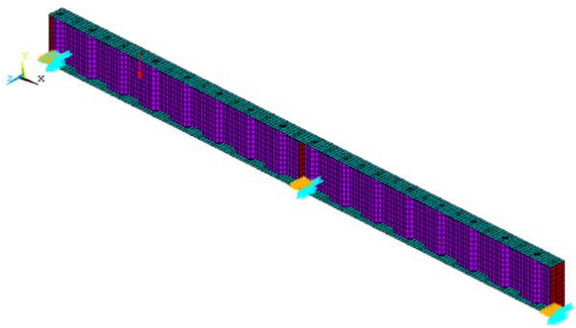


Fig. (10) F.E.M by authors for verification with the experimental model by *Nikolaus L. [15]*.

A finite element model is performed by authors using element 281 in *ANSYS* with the same dimensions; corrugation profile of the web, material properties, and boundary conditions as that considered in the experimental model by *Nikolaus L. [15]* as shown in Fig. (10).

Table (1) shows the comparison between the results of the F.E model and the results of experimental tests by *Nikolaus L. [15]*. It is shown that the values of the patch loading resistances for the analyzed girder which are obtained from the FEM are in good agreement with that obtained experimentally by *Nikolaus L. [15]*, where the differences ranged from 3 to 12.7%.

Table (1) Comparison between results of patch loading resistance from the experimental model by *Nikolaus L. et. al. [15]* and FEM by Authors

Position of applied load	Results of Experimental by Nikolaus L. [15]	Results of FEM by Author	% Difference
A1	219 KN	226 KN	3 %
A2	181 KN	204 KN	12.7 %
B1	188 KN	204 KN	8.5 %
B2	218 KN	234 KN	7.3 %

6. Parametric study

In this research the effect of many parameters in the value of shear buckling resistance are studied for the steel girders with trapezoidal corrugated webs, such as the web thickness; t_w, the web height; h_w, the corrugation angle; Ɵ, and the corrugation width; *a* as detailed in Table (2). The width and thickness of flanges are equal to 500mm and 40mm, respectively for all models. Model [TCW0] represents the original model with the main dimensions of all parameters considered in this study. When the effect of any of these parameters on the shear resistance of these beams is studied, the rest of the other parameters will be of the same dimensions assumed in this model. The effect of the variation in the web thickness from 8 mm to 18 mm is studied through models [TCW8] to [TCW18]. While models from [TCW500] to [TCW3000] give the effect of the variations in the web height from 500 mm to 3000 mm respectively. Additionally, the influence of the corrugation angle; Ɵ is studied by considering two cases for the variation in the corrugation angle. In case {1}, the corrugation depth (d) is assumed to have a constant value, 270 mm, and the increase in the corrugation angle is a result of the decrease of the inclined panel length (c) which is led to increasing the number of corrugation waves along beam span in this case as shown in figure (11) and table (2). While, in case {2}, the length of the horizontal projection of the inclined panel (b) is assumed to have a constant value, 270 mm, and the increase in the corrugation angle is a result of increasing the inclined panel length (c) as shown in figure (12) and table (2).

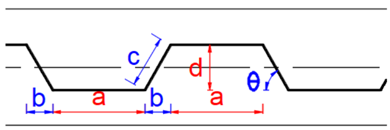


Fig. (11) Web with trapezoidal corrugation TCW1-60

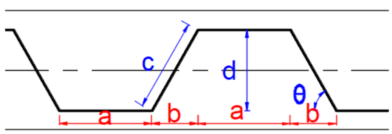


Fig. (12) Web with trapezoidal corrugation TCW2-60

Table (2) The dimensions of the cross-sections for the studied F.E. models (mm, degrees)

Model	h_w	t_w	a	θ	d	b	c
TCW0	2000	12	330	45	270	270	382
TCW-8	2000	8	330	45	270	270	382
TCW-10		10					
TCW-14		14					
TCW-16		16					
TCW-18		18					
TCW-500	500	12	330	45	270	270	382
TCW-1000	1000						
TCW-1500	1500						
TCW-2500	2500						
TCW-3000	3000						
TCW 1-30	2000	12	330	30	270	450	524
TCW 1-45				45		270	381
TCW 1-60				60		155	311
TCW 1-75				75		72	279
TCW2-15	2000	12	330	15	72	270	279
TCW2-30				30	155		311
TCW2-60				60	450		524
TCW-130	2000	12	130	45	270	270	381
TCW-230			230				
TCW-430			430				
TCW-530			530				
TCW-170	2000	12	330	45	170	270	319
TCW-370					370		458
RCW 170	2000	12	330	90	170	0	170
RCW 270					270		270
RCW 370					370		370
ZCW 170	2000	12	0	45	170	170	382
ZCW 270					270	270	
ZCW 370					370	370	

The corrugation angle is considered to increase from 30° to 90° and from 15° to 60° for cases {1} and

{2}, respectively with an increment of 15° for each case. The dimensions of the cross-section for the two cases {1} and {2} are detailed in table (2). The effect of the variation in the corrugation width has been studied by considering it varied from 130mm to 530mm. Finally, the influence of the corrugation shapes on the value of the shear resistance for this type of beam is studied in three types trapezoidal [TCW], triangular [ZCW], and rectangular [RCW] corrugated web with corrugation depth equal to 170, 270, and 370mm.

The results of the comparisons between the values of the shear buckling resistance resulted from the finite element analysis due to the influence of each parameter and the other shear values which were calculated by using the Euro-code equations are illustrated in Figs. from (13) to (20), while the output of the FEA for the eigenvalues for the out of plane displacements (U_Z) for different models are shown in Figs. from (21) to (30).

Figure (13) depicts the values of shear buckling resistance for different web thicknesses from finite element analysis and Euro-code equations. It can be noticed that the shear buckling resistance for steel girders with the corrugated web is increased due to the increase in web thickness. This note is confirmed by Figures (21), (22), and (23) which are shown the eigenvalues for the out-of-plane displacement (U_Z) for girders TCW8, TCW12, and TCW18, respectively. It is found that the eigenvalues for girders with web thicknesses 8, 12, and 18mm equal 0.15E7, 0.257E7, and 0.422E7, respectively. Additionally, Fig. (14) shows that the shear buckling resistance is increased with increasing the web height for plate girders. Figure (15) shows the variation in the values of shear buckling resistance resulting from FEA and EC3 according to the variation in the ratio between the web height and thickness (h_w/t_w). It is clarified that The results of finite element analysis are in good agreement with shear buckling resistance was obtained according to Euro-code equations for different web heights or thicknesses where the difference ranged between 5-7 %. Therefore, the use of EC3, in this case, gives high accuracy results.

Fig. (16) shows that the difference between values of shear buckling resistance obtained by EC3 and FEA increased with the increase in the angle of corrugation (ϴ) where at (ϴ) equal 30°, 45°, 60°, and 75° the shear buckling resistance obtained by EC3 equal 2802, 3113, 3244 and 3244 KN while its value from FEA equal 2955, 3356, 4028, and 6346 KN, respectively. It is observed that the values of the shear buckling resistance resulting from EC3 have the same value for angles (ϴ) equal 60°, and 75° without any change, because Euro-code equations take the largest length of the inclined or horizontal panel in the calculation of shear resistance. On the other hand, the results which were obtained from the finite element analysis clarified that the increasing of corrugation angle leads to an increase in the shear buckling resistance. Therefore, the results of FEA at $\theta > 45$ explain the increasing of corrugation angle in Case 1 leads to a decrease in the inclined panel length (c), and then the number of corrugation waves is increased along the span of the girder and hence the corrugated girder behaves as a girder with a large number of vertical stiffeners as shown in Fig. (17). Therefore, the results of shear buckling resistance calculated by using the EC3 equations may need to be adjusted because the EC3 equations as detailed in Equ. (2) & (3) considered only two modes of failure for the web due to shear either local or global buckling only and neglect the interactive mode failure in its equations. Figs. (24), and (25) show the eigenvalues for the out-of-plane displacement (U_Z) for girders TCW1-30, and, TCW1-60 respectively. It is found that the shape of the formations indicates that the collapse was in model TCW1-30 with (ϴ) equals 30° because of the local buckling in the inclined panel which has a big dimension in this case, but in model TCW1-60 with (ϴ) equals 60° it was because of the interactive mode of failure between the local and global-local buckling.

Figure (18) shows the values of shear buckling resistance which were calculated according to EC3 for girders with different corrugation angles according to *Case 2*. It is observed that there is an inverse relation between shear buckling and corrugation angle, where it equals 2802, 3113, and 3244 KN for angles (ϴ) equal 60°, 45°, and 30°, respectively. While it has the same value for angle (ϴ) equal 15° as that for angle (ϴ) equal 30°. Its value isn't changed according to EC3 because the length of the horizontal panel (a) equals 330 mm which is larger compared with the length of the inclined panel (c) for both girders (CW2-15) and (CW2-30). It is observed that the results of EC3 need further investigations_because the value of shear buckling resistance for girders with small angles as (CW2-15) is higher than for girders with large angles as (CW2-60) because according to the EC3 equation, the shear resistance is controlled by local buckling behavior. While the local buckling behavior is increased in Case 2 due to the increase in (ϴ) depending on the increase in the depth of corrugation (d) with the constant length for the horizontal projection of inclined panel (b) which leads to an increase in the inclined panel length (c). Therefore, the local buckling behavior is governed in the EC3 calculations by the value of (c).

On the other hand, the results obtained from the finite element analysis are clarified that the increase in corrugation angle leads to an increase in shear buckling resistance for all models. The difference between the results of the two methods is because the EC3 equations neglect the effect of the increase in the corrugation depth which gives more stiffness for the web of the girder to deform out of a plane. Figures (26) and (27) show the eigenvalues for the out-of-plane displacement (UZ) for girders (TCW2-30), and, (TCW2-60) respectively. It is found that the values of the out-of-plane deformations of the corrugated web of the girder (TCW2-30) are higher than girder (TCW2-60) because the corrugated depth of (TCW2-60) is higher than (TCW2-30).

Web buckling of the corrugated web is depending on the number of corrugation waves and depth of corrugation. To find out whether the increase in the number of waves or the depth of corrugation is better to increase the web buckling resistance of corrugated web beams, the results of the models for two cases are compared. Figures (24), (25), (26), and (27) plot the web buckling mode and values of eigenvalue for out of plane displacement (U_Z) for plate girders (TCW1-30), (TCW1-60), (TCW2-30), and (TCW2-60), respectively. It is showed that the eigenvalue for girders (TCW1-30), (TCW1-60), (TCW2-30) and (TCW2-60) equal to 0.226E7, 0.308E7, 0.224E7 and 0.340E7, respectively. It also noted that the critical buckling mode for girders (TCW1-30), (TCW2-30), and (TCW2-60) is local buckling, while it is global shear buckling mode for girder (TCW1-60) because this girder has the largest number of corrugation waves. Therefore, the buckling of the web is global for the dense corrugated web.

The influence of the increasing horizontal corrugation width (a) on the value of shear buckling resistance is studied and compared with the results of the Euro-code equations as shown in Fig. (19). It is found that according to the results of the EC3 equations, there is an inverse relationship between the corrugation width (a) and the shear buckling resistance, where it equals 2792, 3001, and 3113 KN for values of (b) equals 530, 430, and 330 mm, respectively. But from the revision of the results of EC3, it is shown that the value of the shear buckling resistance is the same at (a) equals 130 or 230 or 330mm because the calculation of the shear resistance according to EC3 depends on the largest from the inclined panel length (c) or the horizontal panel length (a), which is (c) equals 381mm for all these girders {(TCW-130), (TCW-230) and (TCW-330). On the other hand, the results of FEA are clarified that the increase in corrugation width leads to a decrease in the shear buckling resistance for all models. This is because the increase in corrugation width decreases the number of corrugation waves along the girder span and hence the corrugated girder behaves as a girder with a small number of vertical stiffeners.

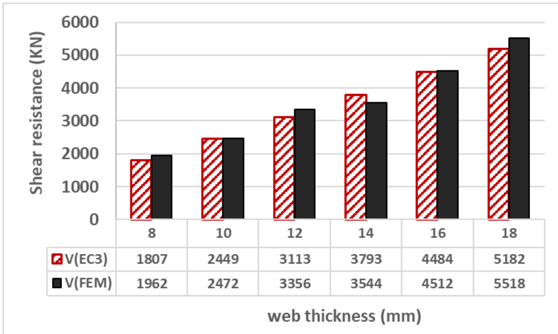


Fig. (13) Influence of web thickness on shear resistance

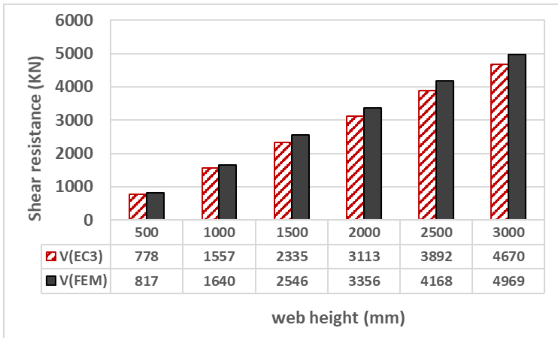


Fig. (14) Influence of web height on shear resistance

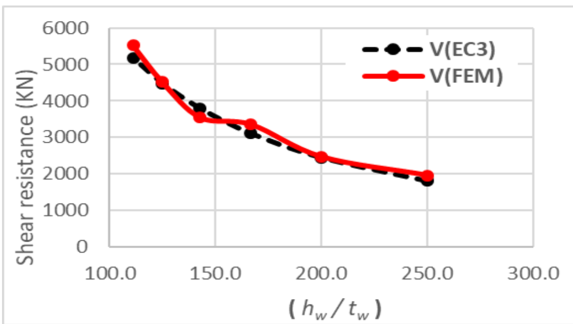


Fig. (15) Comparison of the values of shear resistance from EC3 & FEA due to the variation in the ratio of web height to thickness

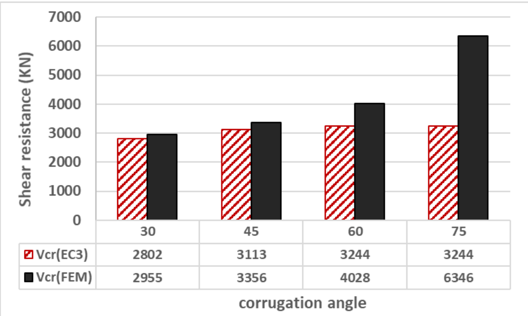


Fig. (16) Influence of corrugation angle on shear resistance (Case 1)

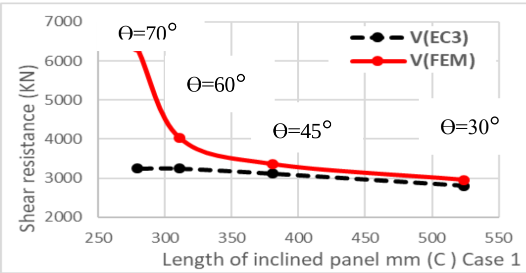


Fig. (17) Comparison the values of shear resistance from EC3 & FEA due to the variation in inclined panel length (c) in Case 1

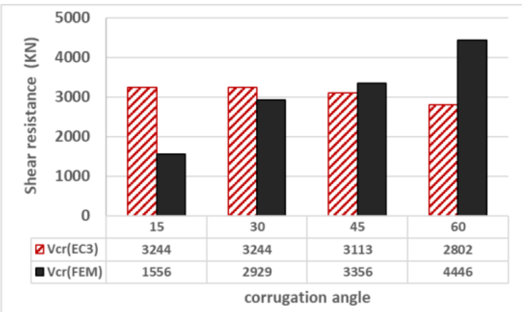


Fig. (18) Influence of corrugation angle on shear resistance (Case 2)

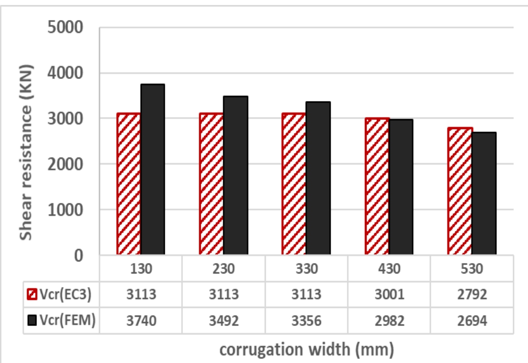


Figure (19) Influence of corrugation width on shear resistance

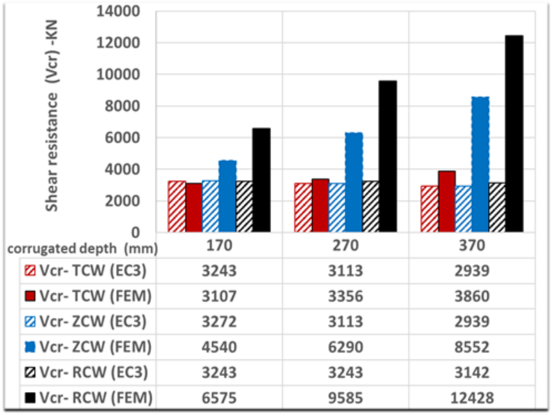


Figure (20) Influence of corrugation shape and depth on shear resistance

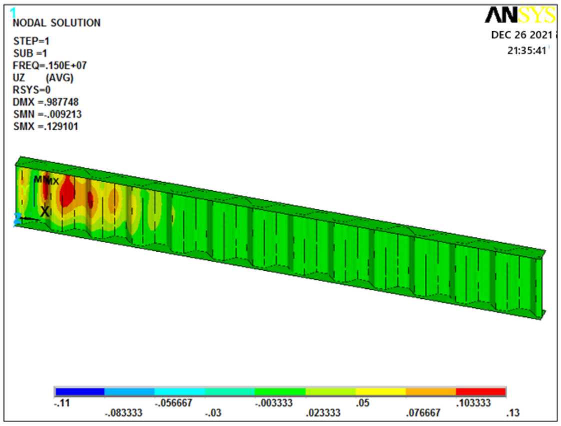


Fig. (21) Web buckling mode for TCW-8

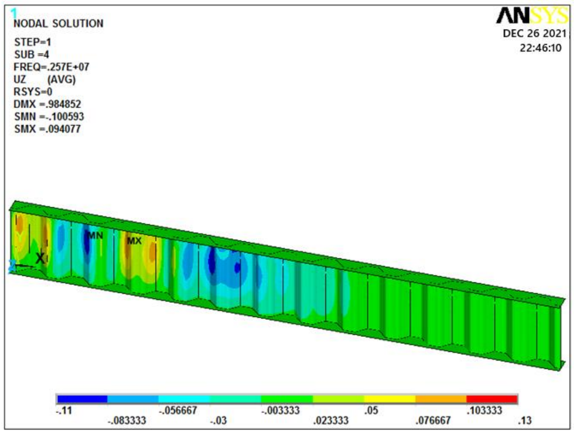


Fig. (22) Web buckling mode for TCW-12

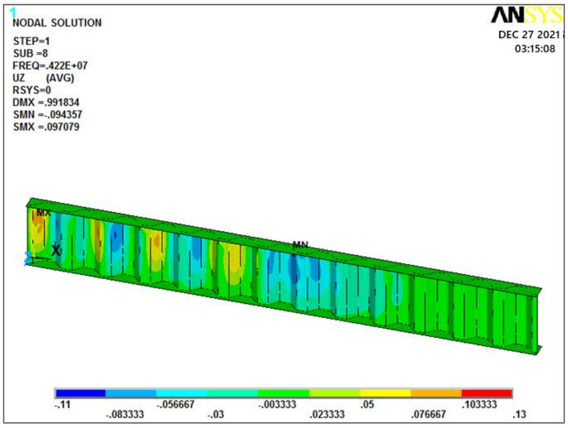


Fig. (23) Web buckling mode for TCW-18

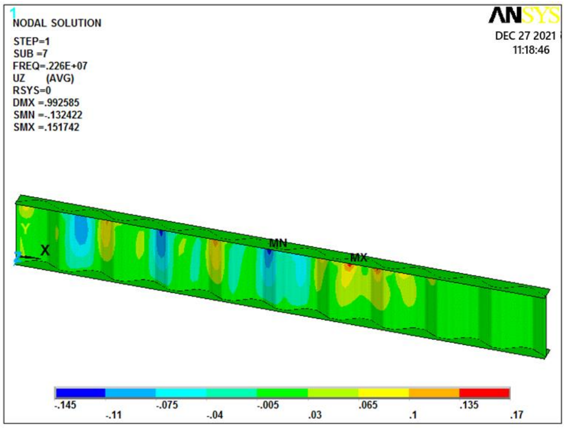


Fig. (24) Web buckling mode for TCW1-30

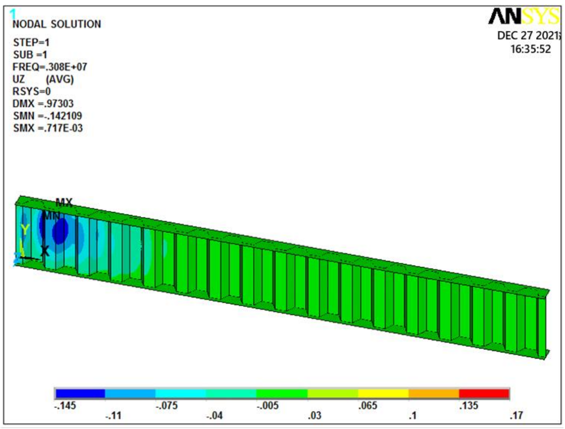


Fig. (25) Web buckling mode for TCW1-60

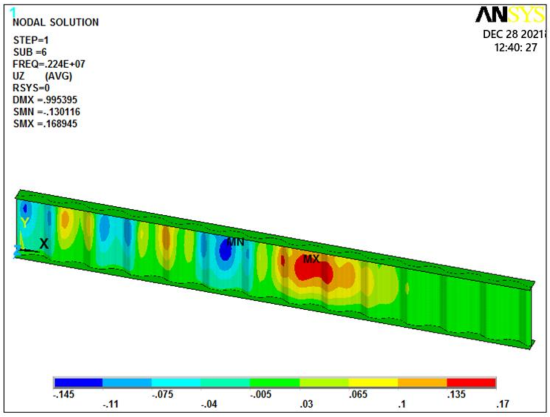


Fig. (26) Web buckling mode for TCW2-30

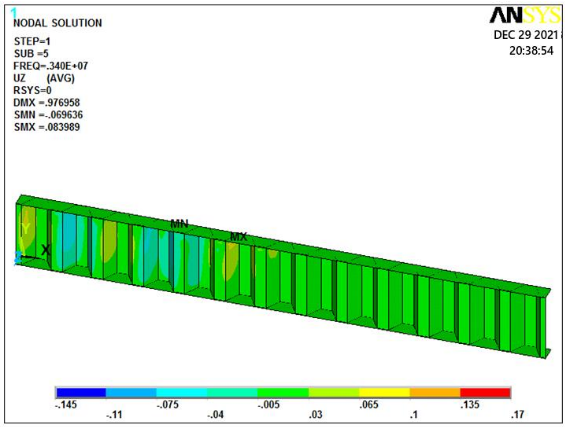


Fig. (27) Web buckling mode for TCW2-60

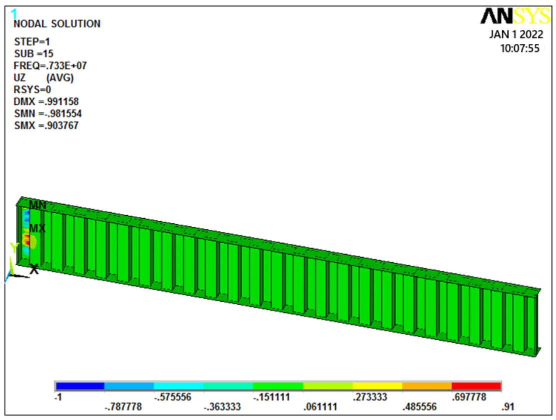


Fig. (28) Web buckling mode for RCW270

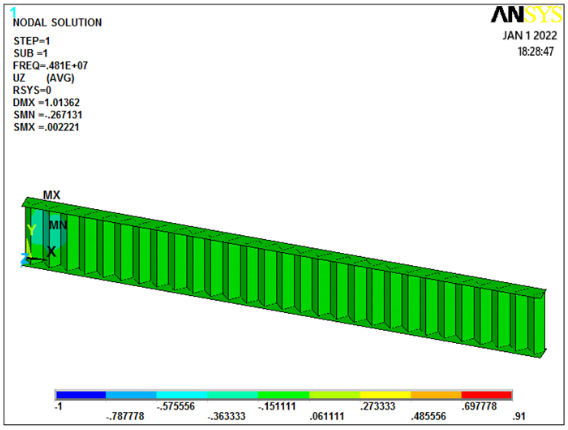


Figure (29) Web buckling mode for ZCW270

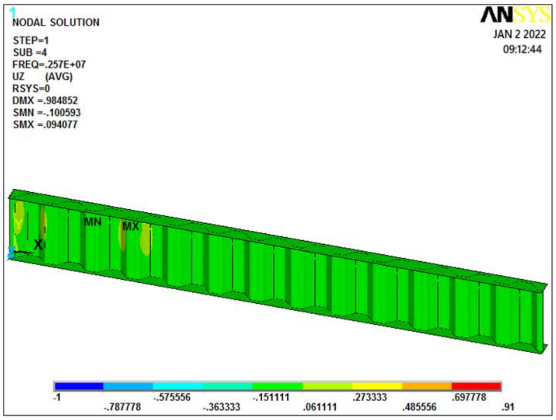


Fig. (30) Web buckling mode for TCW0 (with d=270mm)

Figure (20) presents a comparison between the different results of the FEA and EC3 for the critical shear buckling resistance for plate girders with triangular, trapezoidal, and rectangular corrugated webs. It is shown that according to the results of FEA, the shear buckling resistance for girders with a rectangular corrugated web is higher than for girders with a triangular and a trapezoidal corrugated web. This is because the vertical panels in the case of rectangular corrugated web act as vertical stiffeners. On the other hand, critical shear buckling from F.E.M for girder with the triangular web is higher than that for girders with the trapezoidal web because the number of corrugation waves in the triangular corrugated web is more than that in the trapezoidal corrugated web.

From figure (20) it is observed that the values of the shear buckling resistance calculated according to EC3 for girders (TCW-170), (RCW-170), and (RCW-270) are the same and equal to 3243 KN. It is because the horizontal panel length (a) is the same and is larger than the inclined panel length (c) for these girders. The same observation can be found from the results of girders (TCW-270) and (ZCW-270) where $V_{(EC3)}$ is the same and equals 3113 KN because the largest length for the inclined and horizontal panels equal 381 mm for both girders. Also, the same observation was for the girders (TCW-370) and (ZCW-370). From this, it can be concluded that the European code equations do not differentiate between the different corrugation shapes for the web for the results of the shear buckling resistance.

In addition, the huge divergence between the results of FEA and Euro-code equations clearly appears in cases of the triangular and rectangular corrugated webs. It is found that the shear buckling resistance for models with triangular corrugated webs (ZCW-170), (ZCW-270), and (ZCW-370) according to EC3 equals 3272, 3113, and 2939kN respectively while according to the FEA it equals 4540, 6290, and 8552kN respectively. And also, for models with rectangular corrugated webs (RCW-170), (RCW-270), and (RCW-370) according to EC3 equals 3243, 32433, and 3142kN respectively while according to the FEA it equals 6575, 9585, and 12428kN respectively.

It can be concluded that there is a problem in the use of EC3 equations to calculate the shear buckling resistance due to its condition of choosing the largest of the inclined or horizontal panel in length and do not take into account the number of corrugation waves along the span and the depth of corrugation. So, the accuracy of using the Euro-code equations in the calculation of shear buckling resistance is not accurate and it may need more research handling this point in the case of steel girders with rectangular or triangular webs.

Figures (28), (29), and (30) illustrate the buckling mode for each model and the values of eigenvalue for the out of plane displacement (UZ) for girders (RCW-270), (ZCW0), and (TCW-270), respectively. It can be noticed that the eigenvalue for girders (RCW-270), (ZCW0), and (TCW-270) equals 0.733E7, 0.481E7 and 0.257E7, respectively. It is reached that the girders with the rectangular corrugated web have the highest eigenvalue for out of plane displacement where the vertical panel in the rectangular corrugated web acts as vertical stiffeners and hence the web buckling is decreased. It can be observed that the collapse of girders (RCW-270) and

(TCW0) is due to the local buckling behavior of webs. While the collapse of girder (ZCW-270) is due to the global shear buckling behavior because the number of corrugation waves in the triangular corrugated web is bigger than that of the trapezoidal or rectangular corrugated web.

7- Conclusions

From the analytical study on the web buckling behavior of steel girders with corrugated webs, it can be concluded the following:

1- From finite element analysis, the shear buckling resistance for girders with corrugated webs is increased due to the increase in web thickness, web height, and corrugation angle while it decreased due to the increase of the corrugation width.

2- The shear web buckling resistance of the corrugated web girders increased due to the increase in the corrugation depth and the number of corrugation waves along the girder span.

3- Girders with rectangular webs have shear buckling resistance more than that of girders with triangular or trapezoidal webs with average ratios of 47% and 172%, respectively.

4- The Euro-code equations for the calculations of the shear buckling resistance, sometimes, give conservative results. It is because these equations consider only the largest length of inclined or horizontal panel and neglect the effect of the corrugation depth and the number of corrugation waves along the span of a girder.

5- The results of shear buckling resistance calculated by using the EC3 equations aren't accurate because the EC3 equations considered only two modes of failure for the web due to shear either local or global shear buckling only and neglect the interactive mode failure in its equations.

6- Huge divergence between the results of finite element analysis and Euro-code equations of shear buckling resistance for girders with triangular or rectangular webs.

7- The accuracy of using the Euro-code equations in the calculation of shear buckling resistance may need further adjustments in the case of steel girders with rectangular or triangular corrugated webs.

8- The Euro-code equations do not differentiate between the different corrugation shapes for the web in its results of the shear buckling resistance.

9- The dense, medium, and little corrugated webs collapsed due to the global, interactive, and local shear buckling behavior respectively.

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