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Behavior of Trapezoidal Corrugated Webs Girders with Cutouts: Experimental and Analytical Solution

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Keywords
Corrugated webs, Cut-outs, Experimental test, FEM and ANSYS V20

Abstract

Girders with corrugated webs (GCWs) are nowadays used as structural elements in many applications for their high strength against shear buckling, otherwise cut outs are always provided in these plate elements to enable inspection and servicing. This study presents an experimental and analytical study for investigating the behaviour of girders with corrugated webs (GCWs) having cut outs under shear loading. The experimental program was conducted on three full-scale plate girders, which have been tested under central load at mid-span. The theoretical analysis was conducted using ANSYS V20 to perform a nonlinear technique for the determination of the ultimate shear capacity of the tested girders. Finally, experimental, and finite element model were used to define the ratio of decrement in carrying capacity for corrugated web, having cut out under shear loading. As expected from the ultimate strength, failure mechanism, and load-deflection curves; the results show decrease in the ultimate load carrying capacity with the appearance of one or more cut-outs.

1. Introduction

Girders with corrugated web (GCWs) are composed of corrugated webs that are welded to a pair of flanges, the use of corrugation webs is to increase the out -of -plane stiffness as well as buckling without vertical stiffeners. Corrugated webs were firstly used in aircraft design. Nowadays it is being used in many mega projects like bridges and multistorey buildings see Fig. 1. Several previous studies had been focusing on steel girders with corrugated webs as it was recognized as an excellent load carrying member. In practical life, sections with sinusoidal, trapezoidal, and triangular profiles are most used, as shown in Fig. 2. In structure of industrial and civil buildings for operational reasons, it is necessary to pass ventilation, water supply, heating, electricity, and other

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pipelines, which requires making cut-outs in the beam webs. Thus, beams with corrugated web present an ideal solution is such cases where there is a necessity to have cut-outs see Fig. 3.

Fig. 1. Usage of GCWs in buildings and bridges

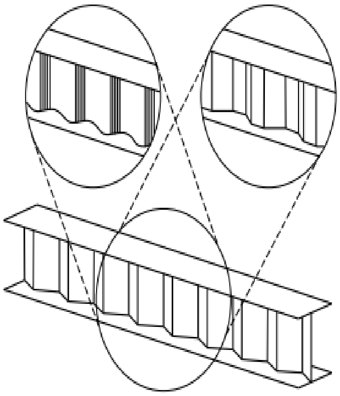


Fig. 2. Girders with sinusoidal, trapezoidal, and triangular web

Fig. 3. Web Cut-outs

Several studies had been concerned on steel girders with corrugated webs. Most of these about the shear and bending behaviour of simply supported beams. Aggarwal, and Papangelis [1] analytically investigated the effect of geometric parameters on the local shear buckling for 90 models of cantilever beams having trapezoidal corrugated webs using ABAQUS. They found that the value of the local shear buckling coefficient KL decreased for beams that were thick, and KL increased with the increase in the ratio of the width panel to the height.

Cao et al. [2] experimentally observed three failure modes with five welded H-shaped steel GCWs; they investigated the buckling performance for the tested girders and compared the results analytically with the ANSYS results. The researchers proved that the capacity due to the shear for the corrugated web was 19.4% for the web thickness equal to 3 mm, where Zubkov et al. [3] experimentally investigated the influence of parameters on the load-bearing capacity of six I-Beams with sinusoidal corrugated web beams under concentrated forces. They found that when the yield strength in the web is achieved, the value of the flanges' tension ranges from 60% to 90%, and the load-bearing capacity is impacted by the eccentricity between the axis of the webs and flanges. Oh et al. [4] Based on the finite element analysis and the mechanical approach confirmed that the accordion effect induced in steel beams with corrugated webs at the prestress transfer was greatly dominated by the geometric characteristics of corrugated webs, and it can be simply quantified using key factors.

El-Amin et al. [5] investigated experimentally and analytically three cantilever beams with corrugated steel webs under shear load, Computer program COSMOS/M 2.8 was used to perform nonlinear analysis to the models' results showed that buckling of the web is local or global for the coarse or dense corrugation, respectively. The effect of having cut-outs on a corrugated web were investigated practically by researchers; they did not receive any design guidance for corrugated steel webs with cut-outs. Lindner and Huang [6] investigated girders with trapezoidal corrugated web plates with cut-outs; they focused on the local buckling behaviour of those girders with web openings. Romeijn et al. [7] presented a basic parametric study for girders with trapezoidal corrugated webs having cut-outs, which showed that any increase in the horizontal eccentricity of the cut out lowers the shear resistance.

Salem et al. [8] are concerned with finite element program COSMOS/M and critical shear load of corrugated web girders with wide web openings with covering parameters. As expected, the openings in shear panels reduced the critical shear load of corrugated web girders. Most models failed by local buckling due to the presence of openings, also an approximate formula for designers have been suggested for practical cases. Samadhan and Laxmikant [9] performed an experimental and parametrical study for steel beams with circular and rectangular web openings; Their results are very helpful for optimizing the aspect ratio and spacing to the diameter ratio of the openings. Kiymaz et al. [10] performed finite element analysis on the series of models of beams with sinusoidal corrugation with different corrugation densities and different relative opening diameters.

2. Shear Failure Modes of Corrugated Steel Webs

Shear failure of corrugated steel webs may occur due to shear yielding, buckling or interactively between yielding and buckling, and can be classified as shown in Fig. 4.

Fig. 4. (a)- Local shear buckling mode; (b)- global shear buckling mode; and (c)- interactive shear buckling mode

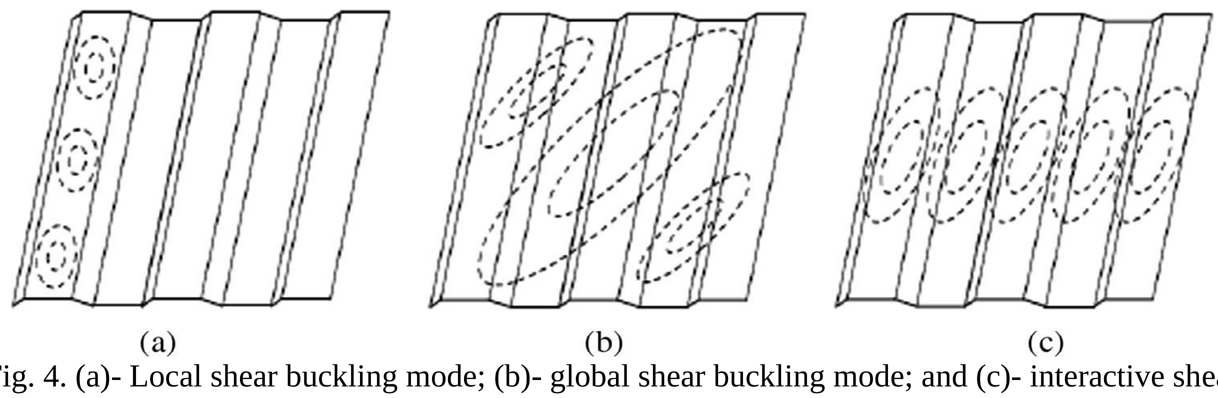


Fig. 4. (a)- Local shear buckling mode; (b)- global shear buckling mode; and (c)- interactive shear buckling mode

2.1 Steel yielding of the Web

Failure of a corrugated steel web may occur by shear yielding, shear stress which causes an element of a corrugated web to yield when it is subjected to pure shear stress state can be determined using Von- Mis's yield criterion with F_y being the yield strength of the steel as shown in Eq. (1):

$$\tau_y = \frac{F_y}{\sqrt{3}} \tag{1}$$

2.2. Stability of corrugated web

Three buckling modes are generally associated with corrugated steel web; namely local, global, and interactive buckling as follows:

2.2.1. Local buckling mode

In this case, the elastic buckling stress $\tau_{cr,L}$ considers these plates as isotropic plates and can be determined by the classical plate buckling theory which is based on research results by numerous investigators including Timoshenko and Gere [11], Bulson [12] and Galambos [13] which is expressed as:

$$\tau_{cr,L} = k_L \frac{\pi^2 E}{12\,(1-\gamma^2)} \left(\frac{t_w}{w}\right)^2 \tag{2}$$

2.2.2. Global buckling mode

In this case, the failure or buckling mode is characterized by diagonal buckling over several corrugation panels. This failure mode is typical for dense corrugation. The buckling stress can be calculated for the whole corrugated web panel, using the orthotropic plate buckling theory; refer to Galambos [13]. However, the global elastic buckling stress $\tau_{cr,G}$ for the corrugated webs has been initiated calculated by Easley [14], as follows:

$$\tau_{cr,G} = k_G \frac{{D_x}^{0.25}\, {D_y}^{0.25}\, E}{t_w {h_w}^2} \tag{3}$$

Where D_x is the transverse bending stiffness per unit length of the corrugated web, D_y is the longitudinal bending stiffness per unit length of the corrugated web and I_y can be calculated as follows:

$$D_x = \frac{q}{s} \frac{E {t_w}^3}{12} \tag{4}$$

$$D_y = \frac{E I_y}{q} \tag{5}$$

$$I_y = 2bt_w \left(\frac{h_r}{2}\right)^2 + \frac{t_w {h_r}^3}{6 \sin\alpha} \tag{6}$$

Elgaaly and Hamilton [15] assumes that k_G is the global buckling coefficient is relatively long compared to h_w and suggest that k_G is to be taken as 31.6 (assuming the web is simply supported by the flanges).

2.2.3. Interaction between all possible failure modes

This includes all possible failure criteria (steel yielding, local and global buckling stresses) which is previously given by Eq. (2) by El-Metwally and Loov [16,17]

$$\frac{1}{\tau_{cr,I}{}^{n}} = \frac{1}{\tau_{cr,L}{}^{n}} + \frac{1}{\tau_{cr,G}{}^{n}} + \frac{1}{\tau_{y}{}^{n}} \tag{7}$$

Where: "n" the exponent of the equation
Where τ_y , $\tau_{cr.L}$ and $\tau_{cr.G}$ are defined by Eqs. (1), (2) and (3) respectively. Eq. (7) gives the least value of the limits of the right-hand side as the maximum limit of the resulting in the lift-hand side, regardless the exponent "n" value. A low value for n (e.g. n=1) gives less than the least of the three limits. On the other hand, higher values for n close to the least of the three limits.

3. Experimental Investigation

3.1. Laboratory Models

Three girders with corrugated girder (GCWs) specimens were manufactured by local steel fabricator in welding workshop, and all tests were performed in the steel construction laboratory, Faculty of Engineering, Assuit University.

3.2. Geometry Models

All girders having web depth of 300 mm, web thickness of 1.5 mm with aspect ratio web height to thickness equals 200, the flange width is 100 mm, and its thickness is 10 mm, and the girder span is 1440 mm. Profiles and dimensions for GCWs are shown in Fig. (5) and Table (1). The cut-outs are always in the first panel behind the support, although the cut-out size is from the centroid of the panel.

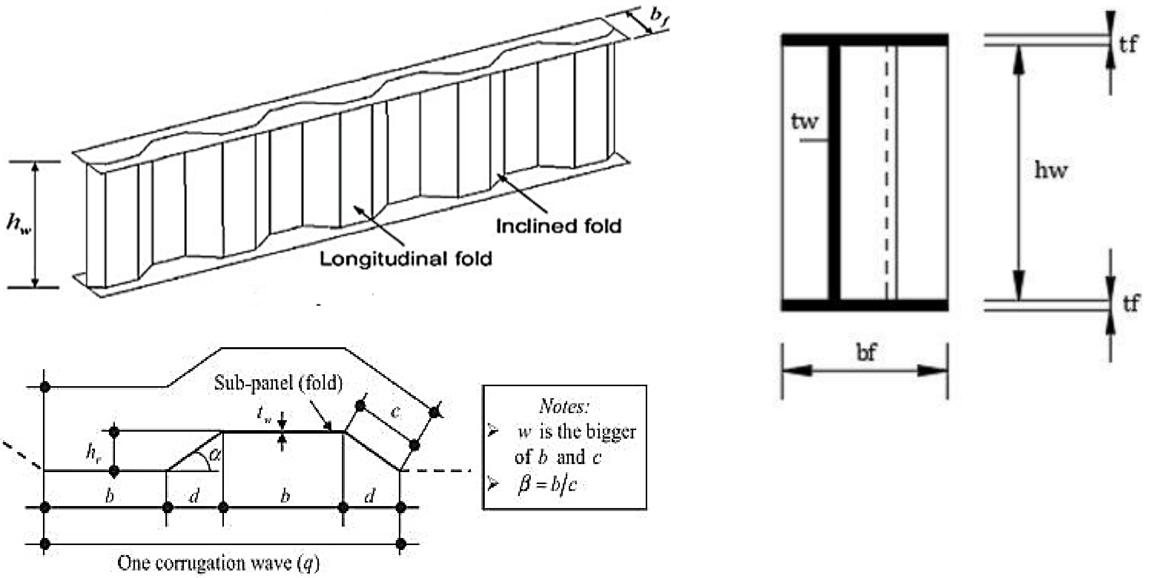


Fig. 5. Dimensions of corrugation profile

Table: 1. Dimensions of girders with corrugated webs

Model	hw mm	tw mm	b mm	d mm	Cut-out size cm	θ degree
GCW 1	300	1.5	100	80	------	37
GCW 2	300	1.5	100	80	15 × 5	37
GCW 3	300	1.5	100	80	2(15 × 5)	37

Three steel-plate stiffeners, 100 mm × 10 mm, were used in GCW1, GCW2, and GCW3: two over the supports and one under the load. All girders were over hanged, and one load was applied at the mid-span of each girder. Webs in all girders were welded continuously to flanges and vertical stiffeners using filet welds. In addition, stiffener plates have filet weld to the flange plates on two sides. Procedures for welding were followed to avoid web imperfection.

3.3. Material Properties

To obtain mechanical properties required for numerical modelling, such as yield stress, ultimate strength, modulus of elasticity, and elongation of tested materials, six tensile coupons were tested, three from the web and three for the flange, which were cut from the tested specimens. Tables (2,3) show the average mechanical properties of the steel used in girders for flange and web respectively under tensile test.

Table: 2. Mechanical properties of flange specimens

Mechanical Prop. of Flange	**Fy MPa**	**Fu MPa**	**E MPa**	**Elongation %**
	281.5	**402.1**	**201000**	**28**

Table: 3. Mechanical properties of web specimens

Mechanical Prop. of web	**Fy MPa**	**Fu MPa**	**E MPa**	**Elongation %**
	262.8	**358.9**	**201000**	**24**

3.4. Test Load

In the steel construction laboratory, Faculty of Engineering, Assuit University. Three simply supported girder were tested, under one concentrated load which was applied at the mid-span of each girder across the top flange and over the mid-stiffener, as shown in Fig. (6) and (7). Fig. (8) represents the two supports for the tested girder, while Fig. (9) shows the load test for the tested corrugated girders. The increasing rate of the load is (0.5t) and it was applied until the beam fails. Linear variable displacement transducers (LVDT) were recorded with computer, as shown in Fig. (10). Visual observations were made during the test. First, for all tests, the loading piston was adjusted to contact the top flange of the distributor beam using suitable number of steel plates.

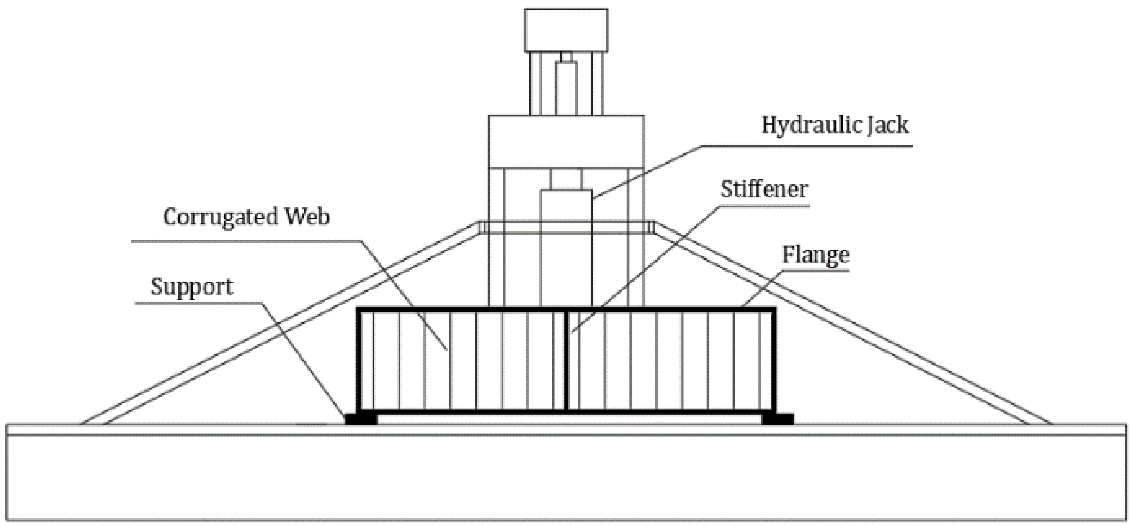


Fig. 6. Schematic view of test setup

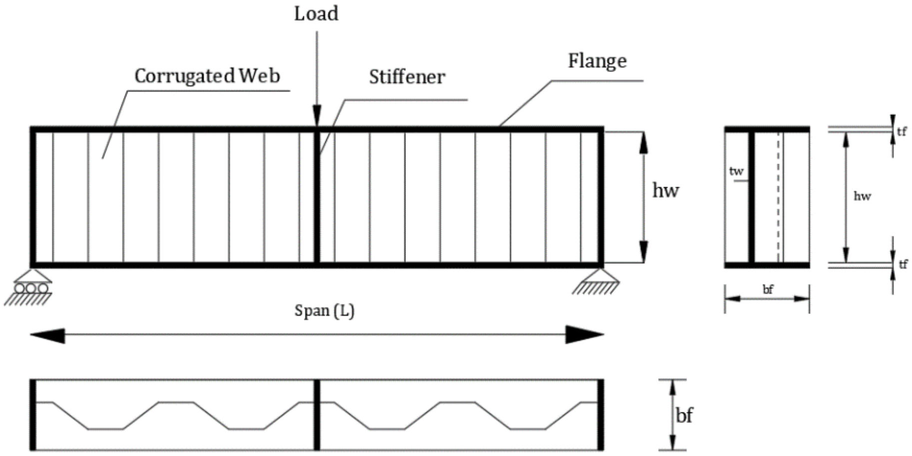


Fig. 7. Load application

a-roller support b-hinged support

Fig. 8. Types of supports for tested girders

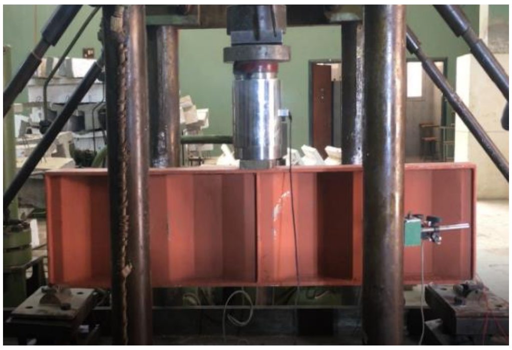


Fig. 9. Corrugated Girder under load test

Fig. 10. Data Logger

3.5. Test Results

Loading of the girders was terminated after the maximum load was attained. Maintaining the load control as the recorded load began to drop was difficult, and ultimate load was reached at the top of load-deflection curve. The test results are given in Table (4). The load increased to the max capacity at failure, for all tested specimens. The relationship between the vertical deflection at mid-height of the web and the ultimate load are plotted in Fig. (11) for GCWs.

Table: 4. Experimental and Finite element Shear load at failure

Specimens No.	Shear load Experimental (kN)	Shear load FEM (kN)	% Difference
GCW1	132.38	136.7	+ 3.2%
GCW2	88.25	91.68	+3.87%
GCW3	72.56	77.46	+6.75%

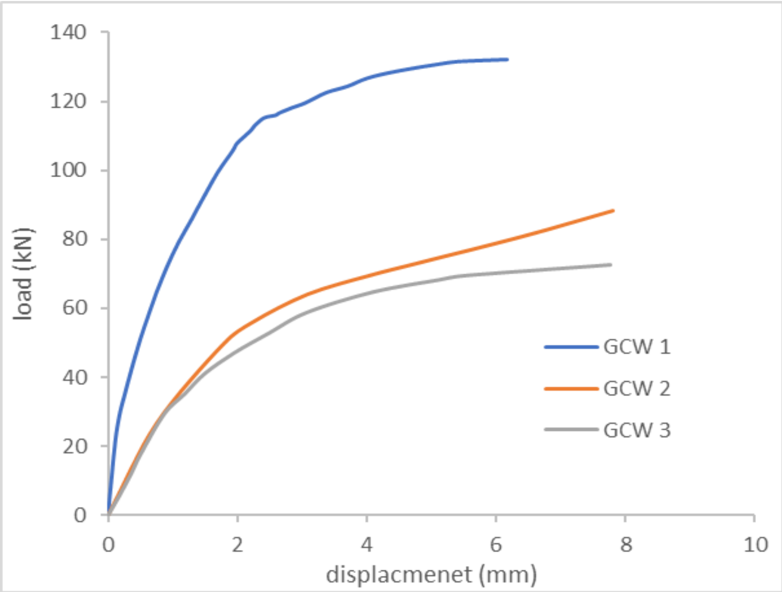


Fig. 11. Applied load versus vertical displacement at mid-span for GCWs

4. Numerical Analysis

4.1. Finite Element Analysis

The finite element analysis package ANSYS Workbench 2020R1 [18] was used to study the behaviours of GCWs for the three tested girder specimens. For members, it is critical to perform buckling analysis and two dependent analyses in the finite element analysis. The first analysis is the eigenvalue buckling analysis for obtaining the buckling modes of the members. The second analysis is a nonlinear analysis that includes both geometric and material nonlinearities. On the ANSYS Workbench platform, it is easy to work on Multiphysics simulations, and a tree-like navigation structure is employed to define the various parts of our simulation as engineering data, geometry, model, setup, solution, and results as shown in Figure 12.

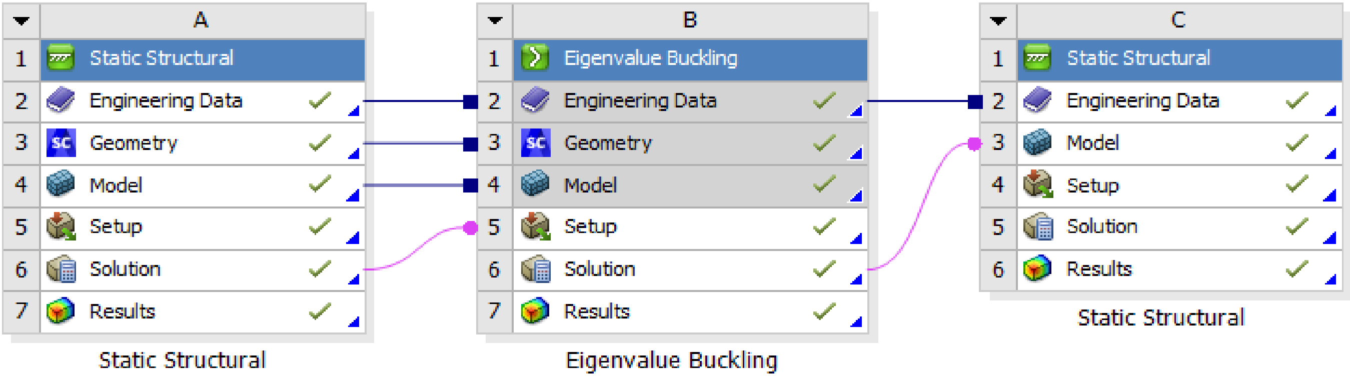
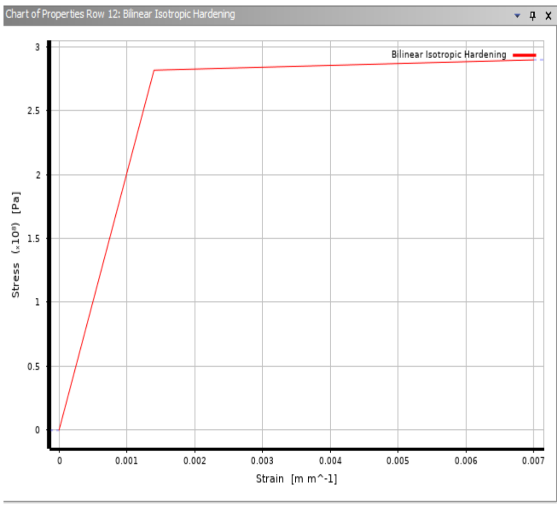


Fig. 12. ANSYS Workbench platform

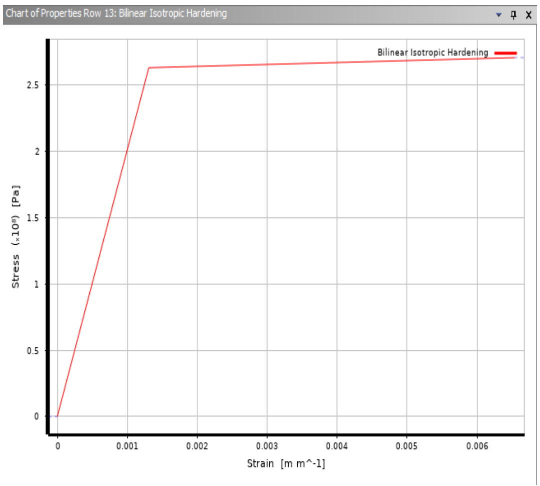
4.2. Modelling Setup

4.2.1. Material and Geometry

Starting with the engineering data, the same mechanical properties for the tested coupons listed in Tables 2 and 3 used in FEM as Bilinear Isotropic Hardening behaviour for both the flange and web, which uses a simplified bilinear stress-strain curves shown in Fig. 13. Then, a 3-D finite element model was created to simulate the test specimens loading and boundary conditions as shown in Fig 14 by using the same geometric dimensions for the corrugated girders listed in Table 1 (see Fig. 15(a)).



(a) Flange stress strain curve



(b) Web stress strain curve

Fig. 13. Stress strain curve for flange and web material

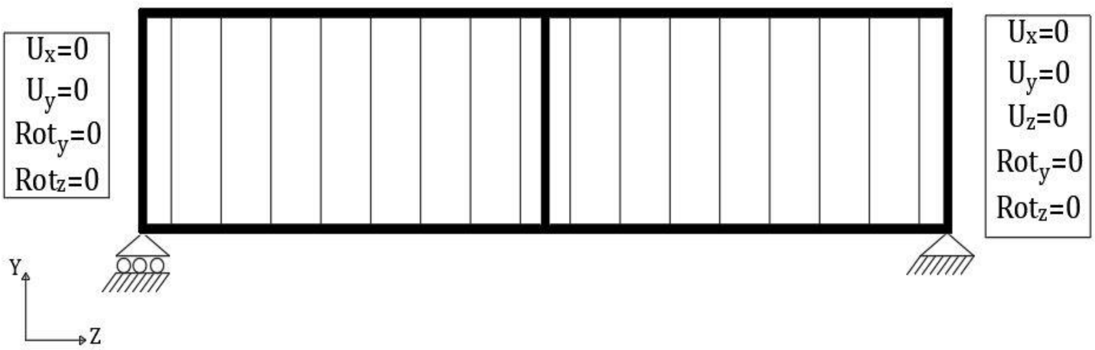


Fig. 14. Schematic view for boundary conditions

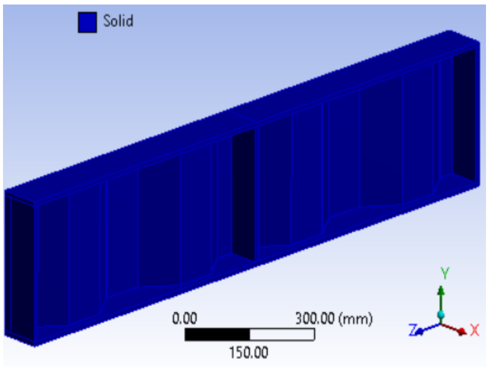


Fig. 15. (a) 3-D specimen model

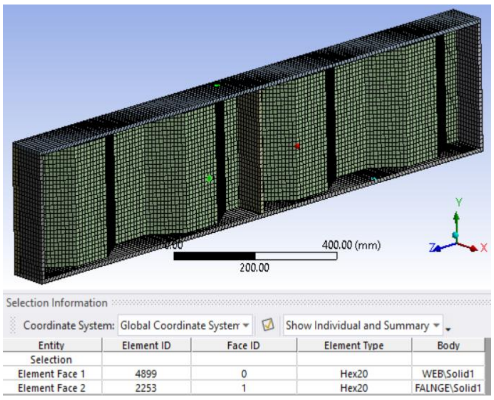


Fig. 15. (b) Specimen mesh

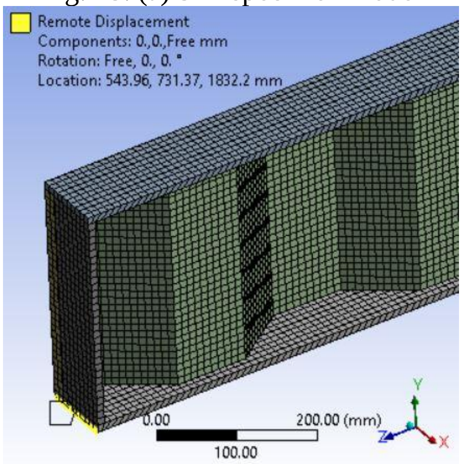


Fig. 15. (c) Boundary condition for roller support

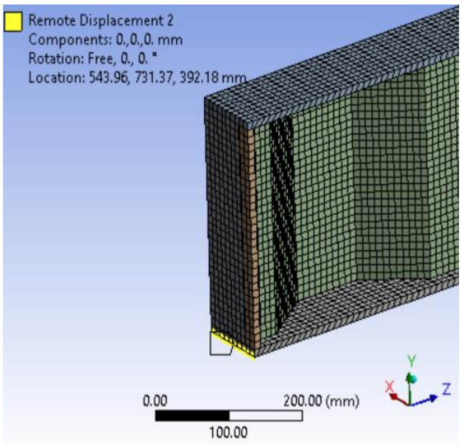


Fig. 15. (d) Boundary condition for hinged support

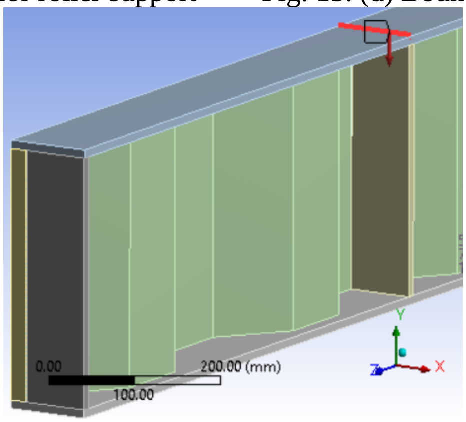


Fig. 15. (e) Specimen loading

4.2.2. Element and Mesh

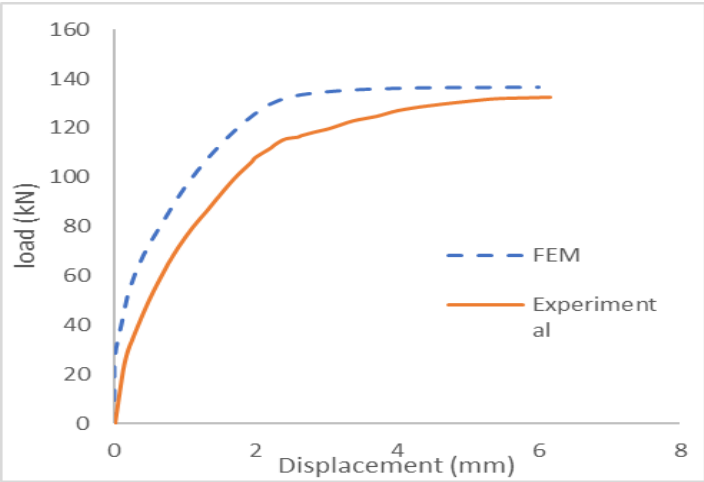
Meshing using ANSYS workbench may be unusual particularly for users being acclimated using mechanical APDL which known as a proper element should be selected by users [19]. However, in the present form, ANSYS workbench automated algorithms to select the suitable type of solid element and for using quadratic 3D solid element with size of 10 mm it automatically choose one of SOLID 187 or SOLID 186 or what called in ANSYS workbench Tet10 or Hex20 respectively, and as we have a regular shape the Hexahedral (bricks) element used to generate a quadratic Hex20 meshing nodes for the solid geometry (see Fig. 15(b)) which have a mid-side nodes with six degrees of freedoms which can follow a curvature in both directions and have mixed formulation capability for simulating deformation of nonlinear materials.

4.2.3. Boundary Conditions and Loads

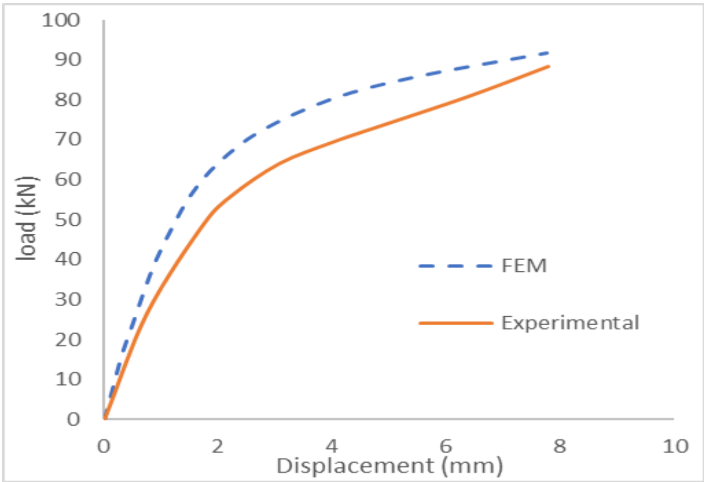
The load test and boundary conditions in FEM analysis simulate the experimental test setup as a simply supported beam under one concentrated load, the supports are from remote displacement edges with six degrees of freedom that prevent Ux, Uy, ROTy and ROTz for simulating roller support and for hinged support we also prevent Uz (see Figs. 15(c) and 15(d)) roller and hinged supports. Lateral torsional buckling was not avoided since there's no instrument within the testing machine to anticipate it, while loads were simulated as a line force at mid span of the tested girder in the gravity direction (see Fig. 15(e)) to obtain the buckling modes and buckling capacity in the stage of eigenvalue buckling analysis then optimize the ultimate carrying capacity for the tested girders using nonlinear analysis.

4.3. Finite Element Results

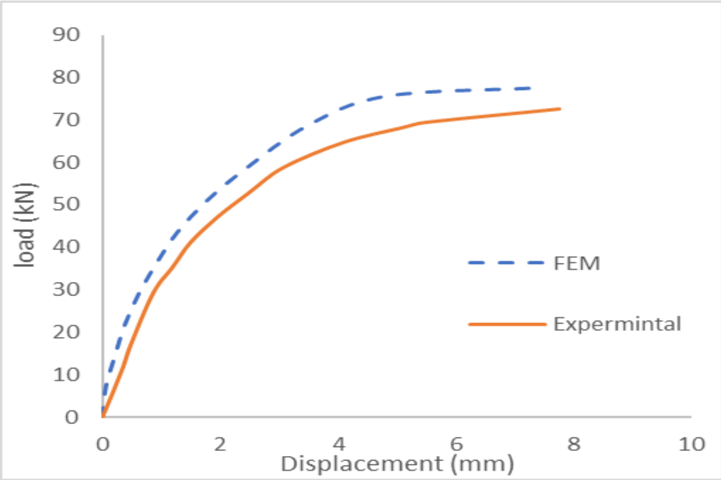
In this section, using nonlinear analysis, we predict the failure modes and the ultimate loads of the tested specimens; we also obtain the relationship between the load and deflection. Table 4 shows the ultimate shear capacity obtained by comparing the experimental test results with the finite element analysis values for GCWs. The comparison showed that the FEM solution was close to the experimental solution. Figure 16 plots the vertical displacement at the midspan of the tested girders versus the applied load for both the experimental and analytical studies.



(a) Load deflection curve for GCW1



(b) Load deflection curve for GCW2

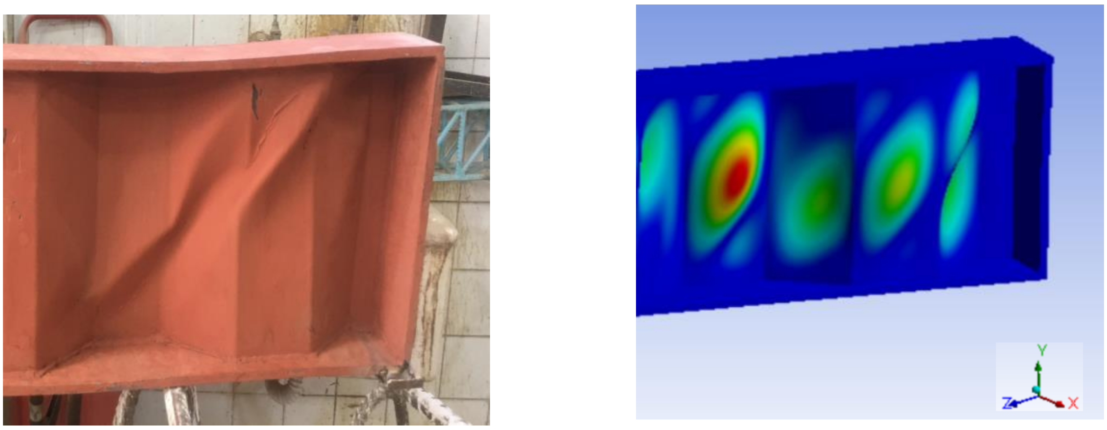


(c) Load deflection curve for GCW3

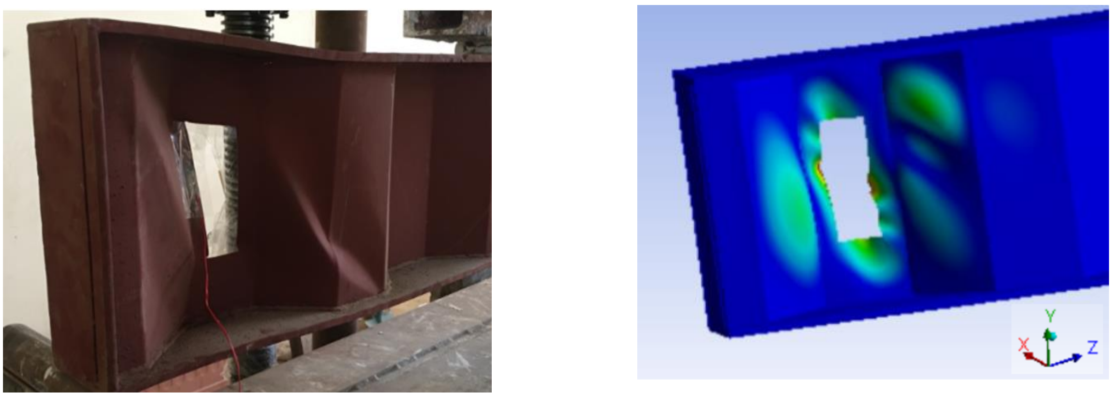
Fig. 16. Load deflection curves for GCWs

5. Results And Discussion

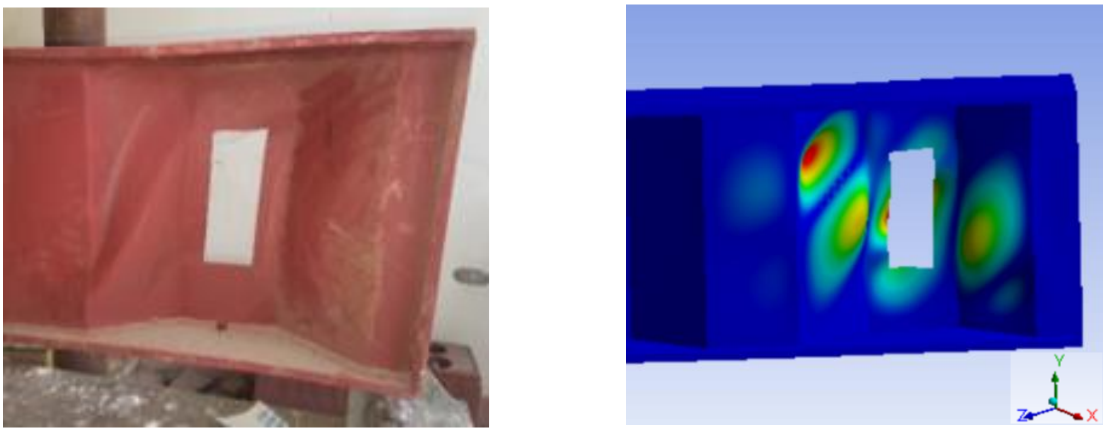
The failure modes of the three girders were due to buckling of web which reached its ultimate capacity without any contribution of flanges the buckling mode shapes for all test specimens at failure are shown in Figs. [17 a, b, c]. In all tested beams, the load was gradually increased until buckling was initiated at the critical buckling load. Then, the load was increased until the final failure occurred at the ultimate load, which was recorded as the load after having extreme deflection.



(a) Buckling mode for GCW1 Experimental versus FEM



(b) Buckling mode for GCW2 Experimental versus FEM



(c) Buckling mode for GCW3 Experimental versus FEM

Fig. 17. Experimental and FEM buckling modes at failure

The ultimate shear capacity resulted from the finite element analysis and the experimental test is presented in Table (4) with good agreement for the girders capacity as shown in Fig. 18.

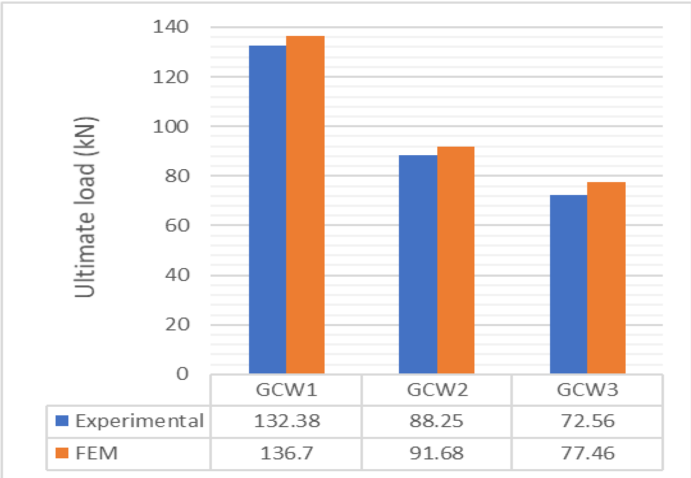


Fig. 18. Results of GCWs

6. Conclusions

From results and comparison, the following conclusions can be drawn:

1. Experimental results show that the failure of the section occurs mainly due to the buckling of web and distortional buckling around the cut-outs.
2. The connection between corrugated web and stiffeners gives efficient joints, because of providing point weld at regular intervals.
3. The comparison of both experimental and analytical results shows that using of Ansys software can predict with a reasonable accuracy the ultimate load behavior of the tested specimens as well as the matching of load-deflection curves.
4. The results show that the ultimate load carrying capacity decreased with the appearance of one or more cut-outs.
5. For girder with one cutout and with the ratio of web thickness of 1.0, GCW2 had a decrease of 33% for ultimate load than GCW1.
6. For GCW3 having two cutouts the ultimate load decrease with 18% than GCW2.

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