

Article

Shear Test of Corrugated Web Girders with Concrete-Filled Compression Tubular Flanges Used in Buildings

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Abstract: Corrugated web girders with plate flanges have been widely applied in buildings and bridges due to the large shear capacity of the corrugated web (CW). However, experiments on corrugated web girders with tubular flanges are limited. Accordingly, this paper explores, through four full-scale small-size experimental tests used on buildings, the shear behavior of a type of girder formed with a CW, concrete-filled tubular flange, and bottom flat plate flange (CWGCFTF) with different CW thicknesses, wavelengths, and concrete strengths. Based on the imprecise results of one current test, a novel simple support device is proposed to improve the accuracy of the shear test of the CWGCFTF. The test results also show that the shear ratios of the tubular flange to the entire cross-section range from 15–18% when the loading reaches that of the corresponding shear stress to 80% of the shear yield strength of the CWs. Moreover, local buckling appears at the top surface of the steel tube with the CW shear buckling failure of the CWGCFTF under the shear tests. At the end, a theoretical equation of the shear ratio of the CW to the whole cross-section is derived, and a shear yield strength equation of the CWGCFTF is proposed and verified by comparisons with the test results.

Keywords: corrugated web girder; concrete-filled tubular flange; shear behavior; experimental test; theoretical derivation



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1. Introduction

Corrugated web girders with plate flanges (CWGPFs) have been widely researched and applied in buildings and bridge structures [1–4], as shown in Figure 1a,b. A three-dimensional diagram of the CWGPFs is extracted and shown in Figure 1c. Because of the weak torsional stiffness of the plate flanges, corrugated web girders with tubular flanges (CWGTFs) have taken the place of CWGPFs to improve the torsion stiffness for some researchers [5–7]. Shear buckling modes of the corrugated web (CW) contain local shear buckling, interactive shear buckling, and global shear buckling [8]. The CW is only subjected to shear forces other than the axial force from bending for its accordion effect [9–11]. The main previous literature on the shear behavior of CWGPFs and CWGTFs is shortly summarized in the following paragraphs.

Previous studies on corrugated web girders mainly contain the shear behavior of the CWGPFs and CWGTFs. Yi et al. [12] conducted a series of finite element (FE) models on the CWGPFs applied in bridge structures to analyze the factors influencing the interactive shear buckling and proposed an equation for the CWGPFs that failed at interactive shear buckling. Sause and Braxtan [13] proposed a shear strength equation for trapezoidal CWGPFs. Nie et al. [14] tested eight trapezoidal CWGPFs under shear and proposed a corresponding elastic shear buckling strength equation on the condition of different failure modes. Hassanein and Kharoob [15,16] analyzed the shear strength of the CWGPFs applied in bridge structures on the real boundary conditions. Leblouba et al. [17] collected shear tests in the literature that tested 12 CWGPFs under shear in the simple support condition and proposed a shear strength equation of CWGPFs. Deng et al. [2] analyzed small-size CWGPFs applied in conventional buildings and proposed a corresponding shear strength

equation by tests and FE simulations. Amani et al. [18] analyzed the shear behavior of stainless steel CWGPFs by tests and FE simulations and found that simulated shear ultimate loads of stainless steel were less than those from the tests of normal low alloy steel when the initial geometrical imperfection value was taken as $h_w/200$. Lee et al. [19] proposed partial CWGPFs and tested the shear failure mode and shear capacity of the girder. Wang et al. [20] analyzed the shear behavior of large-scale CWGPFs that failed at local shear buckling and found that large-scale CWs belong to initial imperfection-sensitive members. As the imperfection value increases, the shear capacity and initial yield strength decrease significantly. The elastic–plastic shear buckling failure modes can be explained as the rigidity bar frame modes. Hu and Wu [21] tested 11 different shear span ratio reinforced concrete beams under shear to analyze the influence of the shear span ratio on the shear capacity. Wen et al. [22] tested 15 corrugated web girders with artificial pits to analyze the shear behavior of the girder. The previous shear strength equation of the CWGPFs is only suitable for the corrugated web girders without pits but not for the girder with pits. More research on the shear strength of corrugated web girders with pits is suggested. Zhang et al. [23] analyzed the post-buckling behavior of corrugated web girders and found that the post-buckling girder seems to be a frame system with a tension web, flanges, and stiffeners. The bending rigidity of the flanges and the width-to-thickness ratio have a great effect on the residual shear strength and collapse mechanism.

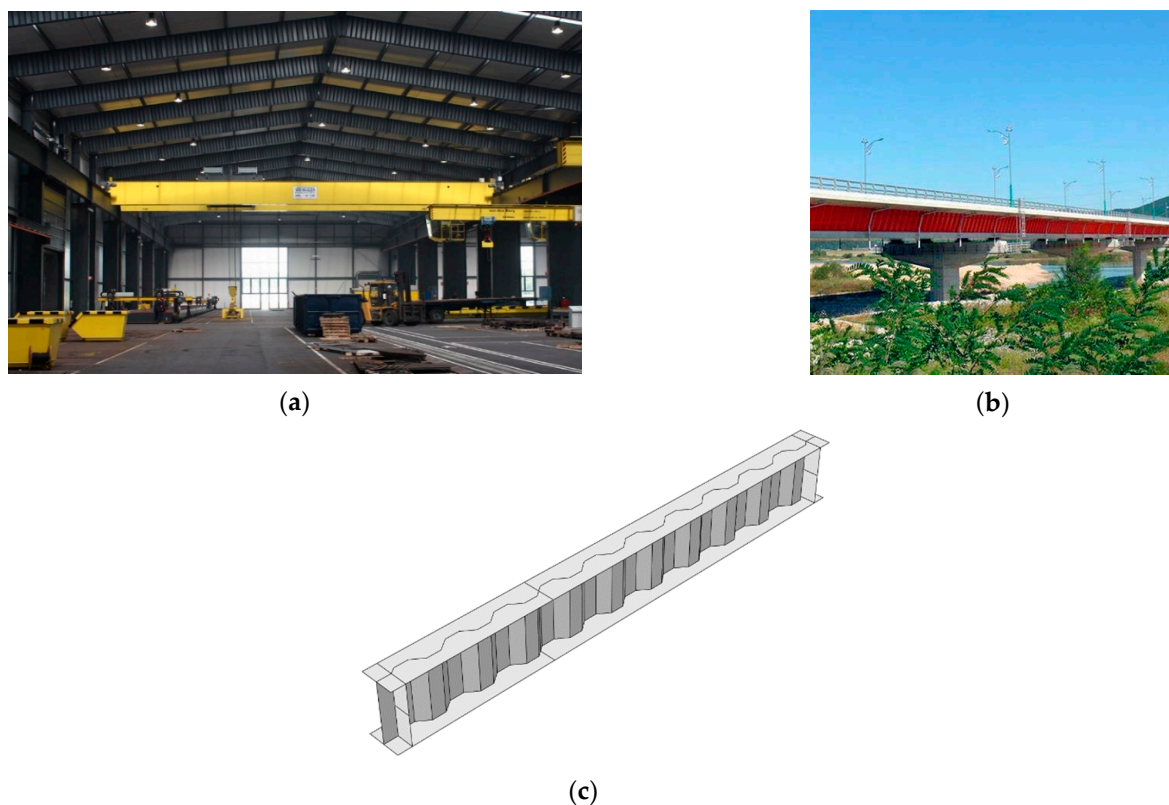


Figure 1. Application of CWGPFs in structures. (a) CWGPFs used in buildings (www.primarystructure.net/corrugated-web-beam, accessed on 1 March 2024). (b) CWGPFs used in bridges (www.researchgate.net/figure/Overview-of-Ilsun-Bridge_fig1_245282124, accessed on 1 March 2024). (c) Three-dimensional diagram of a CWGPF.

Wu et al. [24] analyzed the shear behavior of sinusoidal CWGTFs by bolt connections between flanges and webs and proposed a simple calculation shear strength equation for this girder. Deng et al. [25] analyzed the vertical displacement of the CWGTFs and proposed a deflection equation for this girder on different boundary conditions, which is more accurate compared with the conventional Euler deflection equations. Wang and

Shao [26] tested concreted-filled CWGTFs with big thickness plates on the condition of the ends, where the boundary conditions make the beam satisfy the plane cross-section assumption. Wang and Shao [27] tested concrete-filled CWGTFs under shear and proposed the shear buckling strength equation for this girder. Wang et al. [28,29] analyzed the failure modes of CWGTFs under shear tests, numerical simulations, and a theoretical analysis. Deng et al. [30–32] proposed a new kind of CWGTF and conducted shear tests and FE simulations. The shear distribution ratio of the hollow tube flange to the CW and the shear strength equation of this girder were derived and verified.

Deng et al. [2] and Wang et al. [8] found that the shear behavior of the CWGPFs with small-size CWs applied in buildings is different from those of large-size CWs used in bridges. In this paper, a corrugated web girder with a concrete-filled compression tubular flange is proposed, with a concrete-filled compression tubular flange, CW, and plate tension flange, as shown in Figure 2. The girder is proposed to be applied in structures with little compression at the bottom flange, such as simply supported girders or secondary beams used in buildings. Four full-scale small-size shear tests of the CWGCFTFs with different CW thicknesses, corrugated wavelengths, and concrete strength were conducted. The boundary condition devices used to conduct shear tests and the shear failure modes, shear distribution ratio, and shear capacity of this girder are analyzed. A novel simple support device, based on the weakness of the current test, is proposed to improve the accuracy of the shear test of the CWGCFTF. A theoretical equation of the shear ratio of the CW to the whole cross-section is derived, and a shear yield strength equation for the CWGCFTF is proposed and verified by comparisons with the test results.

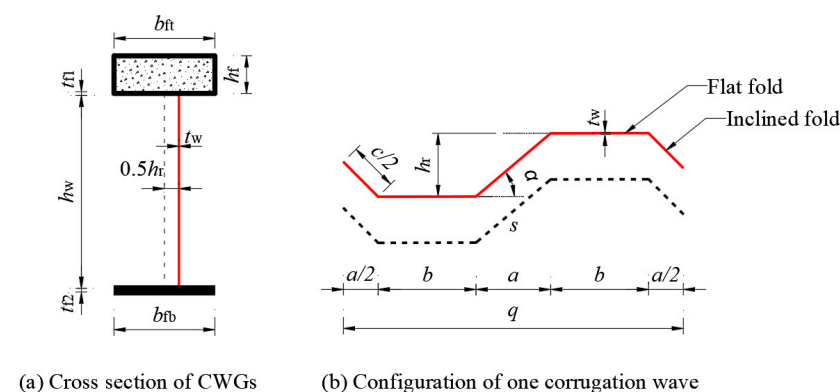


Figure 2. Notation of the CWGCFTFs.

2. Test Design

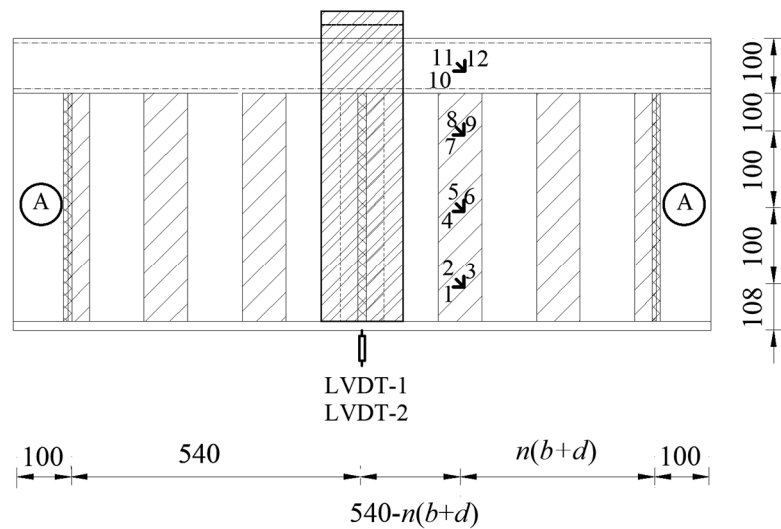
2.1. General Introduction

Four full-scale specimens were tested on their CW thickness, corrugation wavelength, and concrete strength. The dimensions of the top compression tubular flange and bottom tension flange were taken as the same for all the specimens. Test specimens from different sides are presented in Figure 3. It can be seen that the loading location was the mid-span of the specimen with two equal lengths of the CWs, measuring 540 mm and 540 mm. Additionally, the thickness of the tubular flange and bottom flange were relatively large. The height and width of the tube were also large. The dimensions were designed this way to ensure that the specimens fail at the CWs, which was also verified by a finite element simulation. Moreover, supports were considered to extend the nonrigid plates at both ends with a 100 mm length. The middle-span loading plates and bearing stiffener of the test specimens can transmit the load from the horizontal loading plate to the vertical ones in the beginning, then to the bearing stiffener, and eventually to the whole CW height. The test specimens are noted by their CW, corrugation wavelength, and concrete strength. For example, SP-360-2-10 denotes the specimen with a wavelength (q) of 360 mm, CW thickness (t_w) of 2 mm, and concrete strength of 10 MPa. For the corrugation wavelength with a length of 360 mm, the CW dimensions are $a = 80$ mm, $b = 100$ mm, $h_r = 60$ mm,

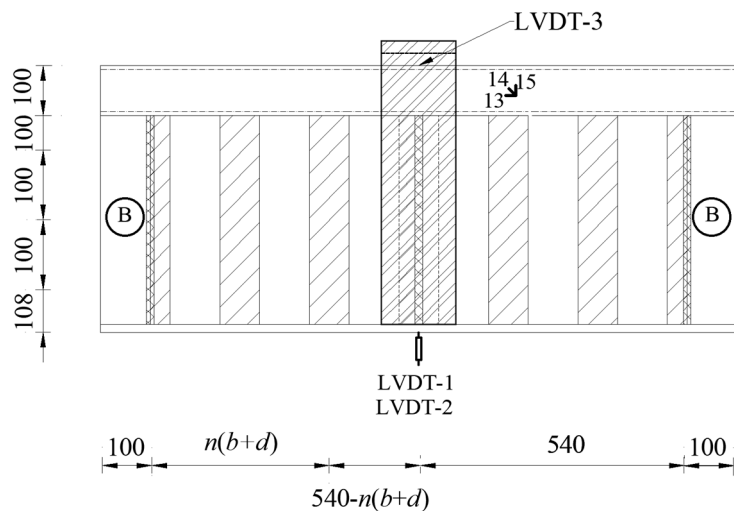
$c = 100\text{ mm}$, and $n = 1.5$. And, for the corrugation wavelength with a length of 180 mm , the CW dimensions are $a = 40\text{ mm}$, $b = 50\text{ mm}$, $h_r = 30\text{ mm}$, $c = 50\text{ mm}$, and $n = 3$. The inclined angle of the CW is $\alpha = 37^\circ$ for all the specimens. For more detailed notations, please refer to the above Figure 2. The notion n is shown in Figure 3a,b. The measured dimensions of the four specimens are shown in Table 1.

Table 1. Measured dimensions of the specimens.

Specimen	b_{ft} (mm)	b_{fb} (mm)	h_f (mm)	h_w (mm)	t_{f1} (mm)	t_{f2} (mm)	t_w (mm)
SP-360-2-10	149.4	150.9	100.5	402	2.57	7.19	1.87
SP-360-2-30	149.8	150.2	100.3	404	2.57	7.19	1.87
SP-180-2-30	149.8	150.3	100.5	405	2.57	7.19	1.87
SP-360-3-30	150.8	150.6	101.4	400	2.57	7.19	2.68



(a) A side view of a specimen.



(b) B side view of a specimen.

Figure 3. Cont.

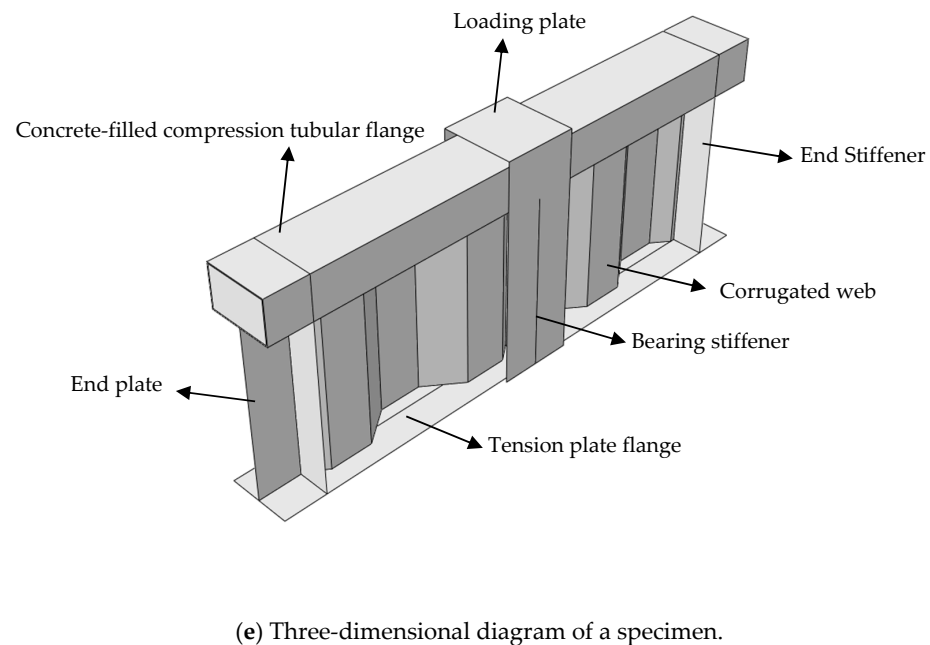
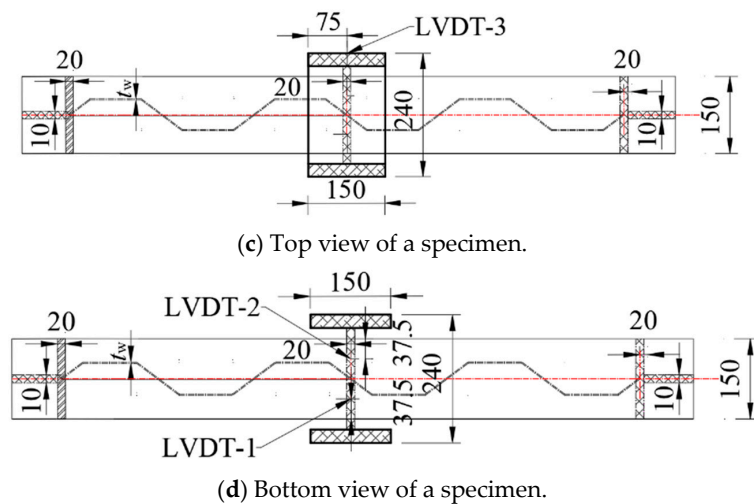


Figure 3. Detailed dimensions of specimens, strain rosette, and LVDTs layout.

2.2. Layout of Strain Rosettes and LVDTs

Deng et al. [31] found that the shear distribution ratio of the tube at the inclined fold is less than that of the flat fold of the CWGTFs. It indicates that the shear force of the CW at the inclined fold of the girder is larger than that of the CW at the flat fold. It is more likely to fail at the inclined fold of the CW under shear. The CWGCFTFs have some shear behavior with corrugated web girders with a hollow compression tubular flange, and the ultimate failure load can be applied to design that of the CWGCFTFs. To analyze the shear distribution rule of the CWGCFTFs and considering the limited static strain gauge channels, three strain rosettes were pasted on the inclined fold at the failure place based on the prefinite element simulation; additionally, two strain rosettes were pasted in the center line of the steel tube at the same cross-section of the CW to eliminate the nonuniformity of the shear distribution caused by the torsion of the specimen. Two vertical LVDTs were designed and positioned on the underside of the bottom flange, named LVDT-1 and LVDT-2, and the average of the measured values was taken as the vertical deflection. The lateral LVDT was designed on the outside of the lateral loading plate to check whether it generated lateral displacement,

and this was performed to ensure that the specimens only fail in terms of shear failure and not in terms of global stability with LVDT-3. Detailed strain rosettes and LVDTs and a three-dimensional diagram of the CWGCFTFs are shown in Figure 3a–e.

2.3. Test Setup

A 500 kN electro-hydraulic loading system was used to conduct the shear tests of the specimens, as shown in Figure 4. The test setup is described as follows: A specimen was placed on two roller supports composed of a steel box and a roller support. The top surface of the steel box had threaded holes, and the bottom of the roller support was connected to the steel box with bolts to fix the roller support, as shown in Figure 5a. The specimen was placed on the steel roller. To establish the clamp support of the girder, thick T-plate threaded holes were used to constrain the lateral displacement of the specimen by tightening the threaded holes with long threaded bars. To ensure the local stability and global stiffness of the T-plate, triangle plates were welded at some distance at both sides of the T-plate, as shown in Figure 4. The top parts of the T-plates were welded by a square connection plate with threaded holes. The square connection plate was connected to a rectangular connection plate tightened by bolts to ensure the global stiffness and stability of the clamp support. The practical test device of both roller supports is shown in Figure 5a. Then, an improved test device contained a roller support and a hinged support, as shown in Figure 5a,b.

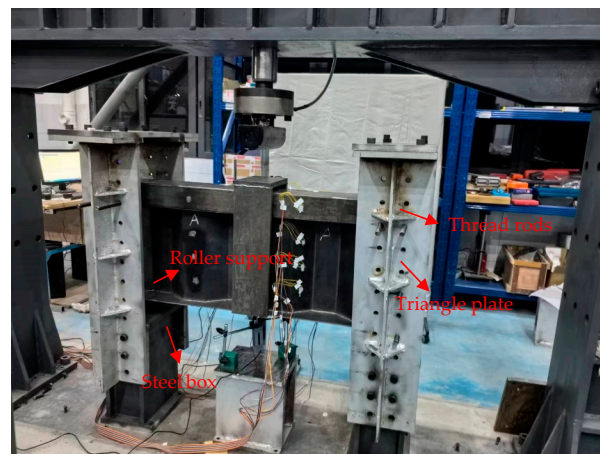


Figure 4. Test device for the specimens.

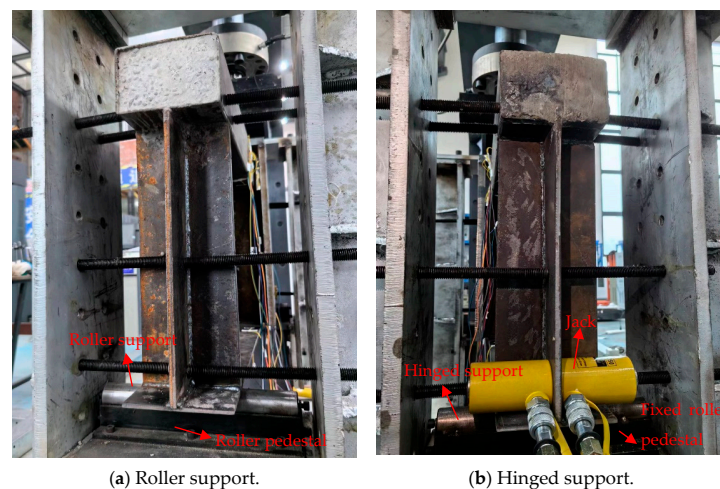


Figure 5. Roller support and hinged support of the specimens.

2.4. Material Properties

To analyze the influence of the concrete strength on the shear behavior of the CWGCFTFs, C10 and C30 concrete cubes were designed before the specimen tests [33]. The blocks are cubes with side lengths of 150 mm and 100 mm for C10 and C30, respectively. Three samples were tested for each concrete cube [34]. The corresponding concrete mix proportion is shown in Table 2. After curing all the samples of concrete blocks for 28 days, they were tested under axial compression by an electro-hydraulic loading system, as shown in Figure 6. The average measured values of the cubic concrete samples of C10 and C30 were 6.4 MPa and 27.0 MPa, respectively.

Table 2. Assumption of different materials of concrete per cubic meter.

Sample	Cement (kg)	Sand (kg)	Stone (kg)	Water (kg)	Water Reducer (kg)
C10	250	644	1307	300	1.25
C30	385	618	1254	209	1.65



Figure 6. Test device for cubic concrete.

Material tests on steel coupons were also performed before the tests of specimens to analyze the effect of real steel strength on the shear behavior of the CWGCFTFs. The coupons were made of four measured thicknesses of the steel plates, including CWs with two different thicknesses, tubular flanges, and plate flanges. Tensile coupons were designed referring to the specifications (GB/T 228) [35]. The measured yield strength, ultimate tensile strength, and elastic modulus are presented in the following Table 3. The coupon test device is also an electro-hydraulic loading system, as shown in Figure 7.

Table 3. Result of steel coupon tests.

t (mm)	E (GPa)	F_y (MPa)	F_u (MPa)	ε_u (%)	F_u/F_y
2.57	200.12	307.3	390.1	20.7	1.27
1.87	192.86	326.5	403.2	22.8	1.50
2.68	203.72	316.7	434.7	21.4	1.37
7.19	207.2	299.1	449.7	20.9	1.23



Figure 7. Test device for steel coupons.

2.5. Initial Imperfection Measurement

The shear buckling failure of the CWs is sensitive to the initial geometrical imperfection [14], namely the out-of-plane offset of the corrugated webs. Accordingly, it is essential to measure the actual initial geometrical imperfection magnitudes of the CWs prior to the specimen tests. The initial geometrical imperfection is measured by two rigid rulers, where one is positioned next to the tube side and the bottom flange vertically and the other rigid rule is used to measure the distance between the outside of the flat fold of the CW and the former ruler. The errors in the distance with the designed dimensions are the initial geometrical imperfection magnitudes of the CWs. The initial imperfection values of the specimens on both sides (A side and B side) were measured at the designed positions of the flat folds provided in Figure 8. The maximum measured imperfection values for SP-360-2-10, SP-360-2-30, SP-180-2-30, and SP-360-3-30 were 4 mm, 6 mm, 4 mm, and 5 mm, respectively.

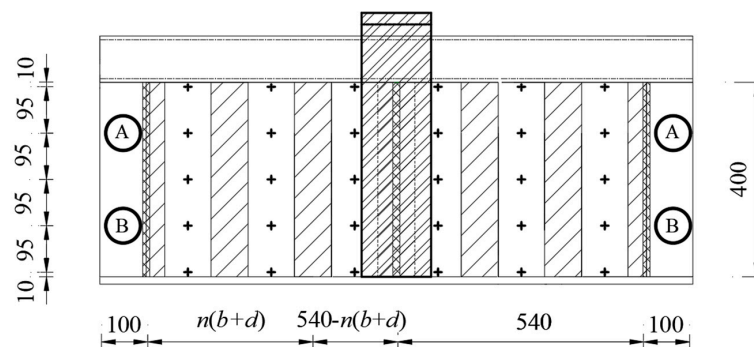


Figure 8. Designed positions when measuring initial geometric imperfection.

2.6. Loading Procedure

The whole loading procedure was as follows: First, place a specimen on the supports. Vertical and lateral constraints were used on the end supports to constrain its lateral displacement. Secondly, check that the inclination of the horizontal loading plate is small with a level instrument to ensure that the actuator can transfer its load vertically to the specimen. Thirdly, preload the specimen to 20% of the ultimate load as presimulation to ensure that the strain rosettes and LVDTs work correctly and also to delete the error of bearing clearance. The formal loading process controlled the displacement of all the specimens at 0.5 mm per minute. Loading was also applied to the specimen to record

the entire failure process after it reached the ultimate shear capacity and deformation obviously occurred.

2.7. Samples Fabrication

The specimen is made of a steel part and a concrete part. The steel part is welded by welding the hollow tubular flange to the CW, the bottom plate flange to the CW, and other stiffeners to the steel plate connection via CO₂ gas-shielded welding. The welded steel segment is shown in Figure 9a. After the welding of the steel, the steel part is positioned vertically, where the bottom surface of the tube is sealed with a high-toughness tightened bag and the mixed concrete is poured into the tube artificially. The pouring concrete procedure is slow, and the concrete is vibrated by using a concrete-vibrating spear to increase the compactness of the concrete, as shown in Figure 9b.



Figure 9. Samples fabrication.

3. Test Results

3.1. Lateral Displacement of the Specimens

The measured maximum lateral displacement of the specimens was less than 1 mm, except for SP-60-2-20, as shown in Figure 10. The maximum lateral displacement of SP-60-2-20 was about 2 mm, and this was caused by the incline of the horizontal loading plate. After checking the measured vertical displacement values of LVDT-1 and LVDT-2 at the two points of the bottom surface at the tension plate flange, they were almost the same. This means that the lateral displacement of the loading plate had little effect on the analysis of the shear behavior of the specimen. The specimen only failed in terms of shear failure, which satisfied the requirement of a single-failure control.

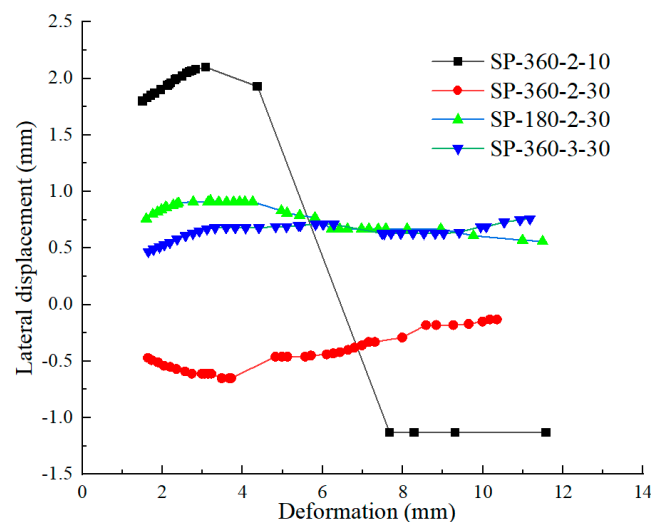


Figure 10. Lateral displacement–deflection relation of the tests.

3.2. Improvement in Boundary Condition

For the specimen SP-360-2-20, shear buckling occurred at two inclined folds and flat folds, as shown in Figure 11a. The specimen suddenly moved toward the web buckling position heavily with a loud sound. To analyze this phenomenon, the authors simulated the same model on two roller supports via displacement loading with different values. The FE results showed that the ultimate loads were largely different as the value of the displacement loading changed. This indicates that the two roller supports at both ends were not suitable for the analysis of the shear behavior of the girder. When the boundary condition in the FE simulation was replaced by a hinged support and a roller support, the results from the FE simulation almost agreed with the test results of the specimen.

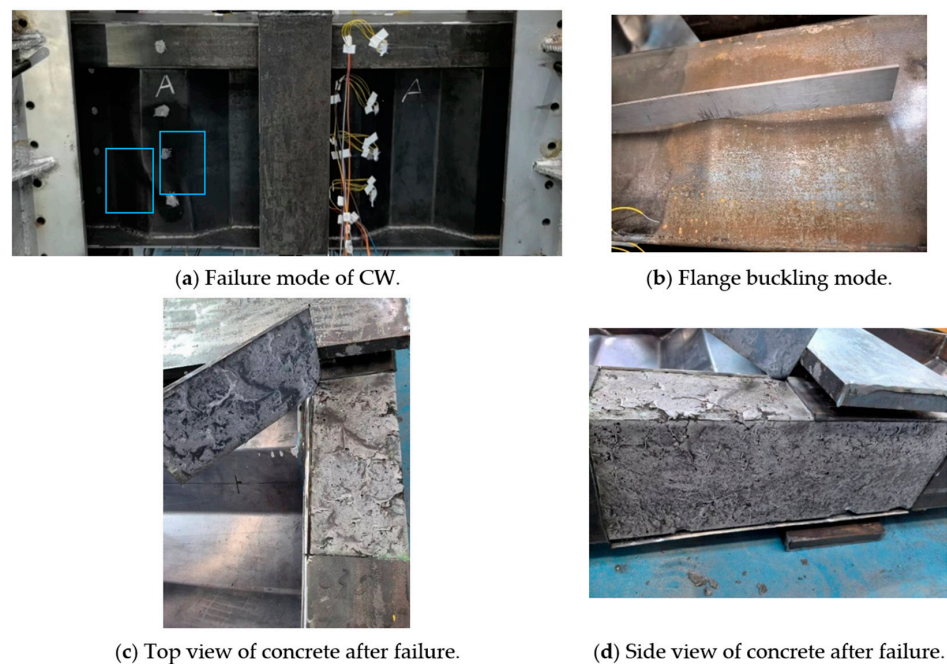


Figure 11. Failure modes of SP-360-2-10.

To improve the boundary condition device of the simple supports, the authors proposed a novel boundary condition device made of a roller support to constrain its vertical line displacement at one end and a hinged support with jacks to establish the actual boundary condition to constrain the horizontal line displacement and vertical line displacement at the other end. Detailly, the steel roller was welded with the roller pedestal with intermittent welding to keep it fixed. Two 100 kN jacks were installed at the elongated end plate and threaded rods, as shown in Figure 5b, to generate a friction force along the length of the specimen.

3.3. Shear Failure Load and Modes

The test results on the shear behavior are described herein. According to Hooke's law, the yield shear strength (τ_y) was calculated by Equation (1). The yield shear load (V_y) was calculated by determining the shear area (A) and multiplying the shear stress as per Equation (2). The shear area of the CW was the cross-section area of the web. The shear area of the steel tubular flange consisted of two side areas of the tube, and the shear area of the concrete was the whole cross-section area of the concrete. f_w means the shear yield strength of the CW. The calculated shear forces and loads of the CWGCFTFs are shown in Table 4. The notions are described as follows: f_{Tu} means the shear yield strength of the tube. A_w means the cross-section area of the CW. A_{Tu} means the cross-section area of both lateral sides of the tube. V_w means the shear yield force of the CW. V_{Tu} means the shear

yield force of the tube. N_U means the ultimate load of the specimen. V_U means the ultimate shear force of the specimen:

$$\tau_y = f_y / \sqrt{3} \quad (1)$$

$$V_y = \sum \tau_y \times A \quad (2)$$

Table 4. Shear behavior of different components.

Specimen No.	f_w (N/mm ²)	f_{Tu} (N/mm ²)	A_w (mm ²)	A_{Tu} (mm ²)	V_w (kN)	V_{Tu} (kN)	N_U (kN)	V_U (kN)
SP-360-2-10	189	177	748	515	141	91	293	146
SP-360-2-30	189	177	748	515	141	91	294	147
SP-180-2-30	189	177	748	515	141	91	307	153
SP-360-3-30	183	177	1073	515	196	91	423	212

As for the load–deformation curve of SP-360-2-10, when the load of the specimen reaches 293 kN, the corresponding vertical displacement is 3.1 mm. Before the ultimate point, the slope of the load–displacement is almost constant, as shown in Figure 12. The shear force relation of the components and the load is presented in the above Table 4, from which it can be seen that the web yielded for shear (141 kN) before the failure of the web shear buckling (146 kN). According to Sause and Braxtan [13] and Hassanein and Kharoob [15], stocky CWs cannot reach the shear yield strength. The influence of the concrete-filled compression tubular flange in sharing the shear capacity occurs. As the load increases, the stiffness of the girder decreases obviously. Then, the load suddenly drops down to 160 kN, and the corresponding vertical displacement is 4.4 mm. This indicates that the plasticity of the girder is deficient.

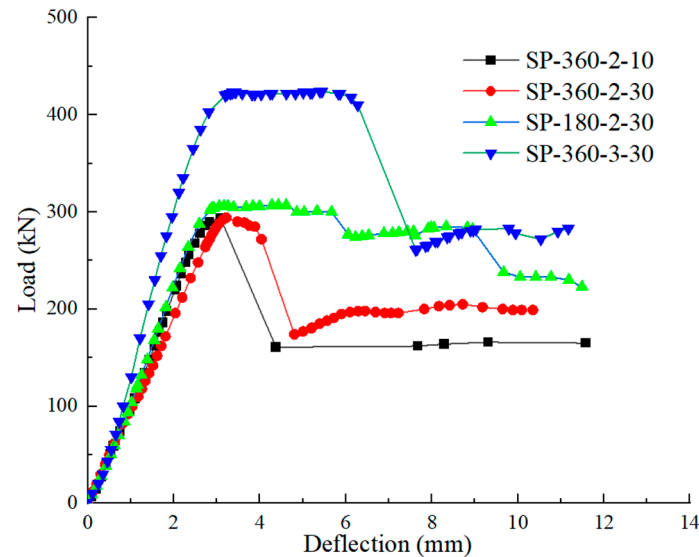


Figure 12. Load–deflection relation of the tests.

Figure 11a shows the failure position of the corrugated web of SP-360-2-10. Two shear failure buckles in the 45° direction of the folds are marked with a blue rectangle. It belongs to the interactive shear buckling. Afterward, the filled concrete tubular flanges failed at local buckling under the action of the bending, which made the top surface of the steel tube part away from the inner concrete, as shown in Figure 11b. Cutting the parts of the steel tube out, the inner concrete is undamaged and does not show crush failure, as shown in Figure 11c,d.

When the load of SP-360-2-30 reaches 294 kN, the corresponding deflection is 3.2 mm. Before the ultimate point, the slope of the load–displacement does not change, as shown

in Figure 12. As presented in Table 4, it can be calculated that the web yielded for shear (141 kN) before the failure of the web shear buckling (147 kN). Therefore, the concrete-filled compression tubular flange also shares the shear capacity. As loading continues, the stiffness of the girder decreases greatly. Then, the load suddenly drops to 174 kN and the corresponding deflection is 4.8 mm. This indicates that the plasticity of the girder is also deficient.

The CW shear buckling failure of SP-360-2-30 is shown in Figure 13a. Shear failure buckles of folds appear in the direction of 45° . This is the interactive shear buckling. The steel tube also fails at local buckling under compression, as shown in Figure 13b. Cutting the parts of the steel tube out, the concrete is integrated without compression failure, as shown in Figure 13c,d.

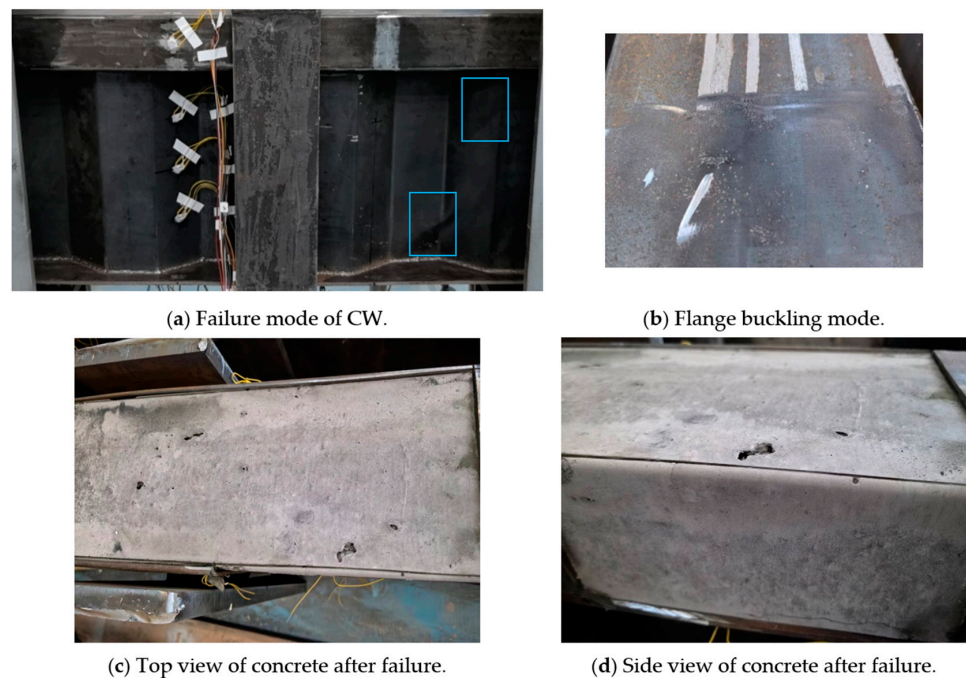


Figure 13. Failure modes of SP-360-2-30.

Before the ultimate load of SP-180-2-30 reaches 307 kN, the corresponding deflection is 3.2 mm. The corresponding slope of the load–deflection curve is nearly the same. The stiffness of the girder decreased gradually with the increase in the load. As the displacement loading continued, the load dropped down to 282 kN little by little, and the corresponding deflection reached 8.9 mm, as shown in Figure 12. This indicates that this is a plastic failure.

Figure 14a shows the failure position of the corrugated web of SP-180-2-30. Two folds both failed at shear buckling. This was also the interactive shear buckling. The filled concrete of the tubular flanges failed at local buckling in Figure 14b. The top side and lateral side of the concrete were also undamaged, as shown in Figure 14c,d.

Before the load of SP-360-3-30 increased to 423 kN, the corresponding deflection was 3.4 mm. The slope of the load–deflection curve did change. As the load increased, the stiffness of the girder decreased gradually. As the displacement loading continued, the load dropped down to 410 kN, and the corresponding deflection reached 6.3 mm, as shown in Figure 12. This indicates that this is a plastic failure.

Figure 15a shows the failure mode of SP-360-3-30. Two shear buckling failures of folds appeared in the CW. This is the interactive shear buckling. The steel tube of the compression tubular flange failed at local buckling under the bending, as shown in Figure 15b. Seen from the top side and lateral side of the inner concrete, it is unbroken, as shown in Figure 15c,d.



Figure 14. Failure modes of SP-180-2-30.

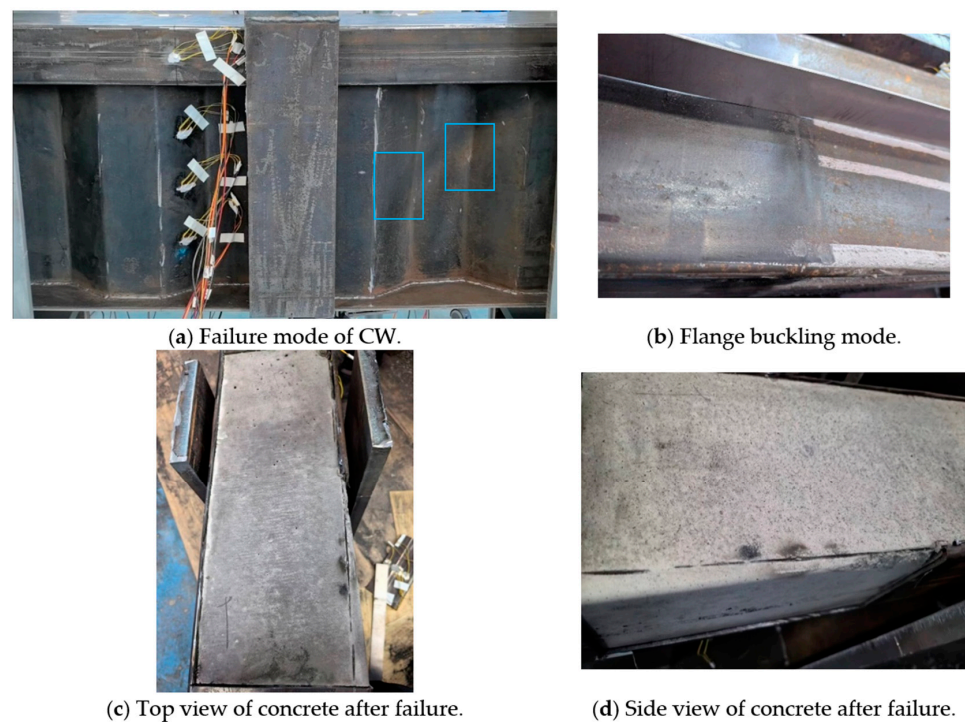


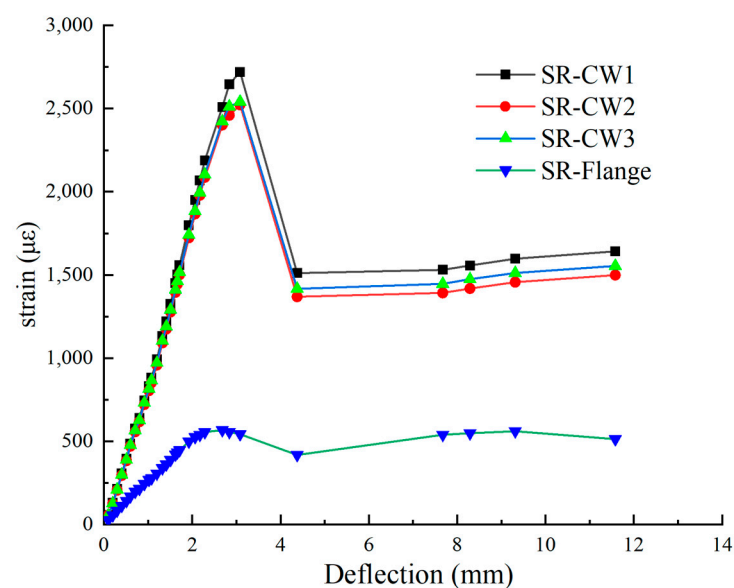
Figure 15. Failure modes of SP-360-3-30.

To summarize, regarding the failure modes of the four specimens, it can be seen that they all failed at shear buckling in terms of the CW, whereby the steel tubular flange failed at compression buckling, and the inner concrete of the steel tubes of the CWGCFTFs was not broken.

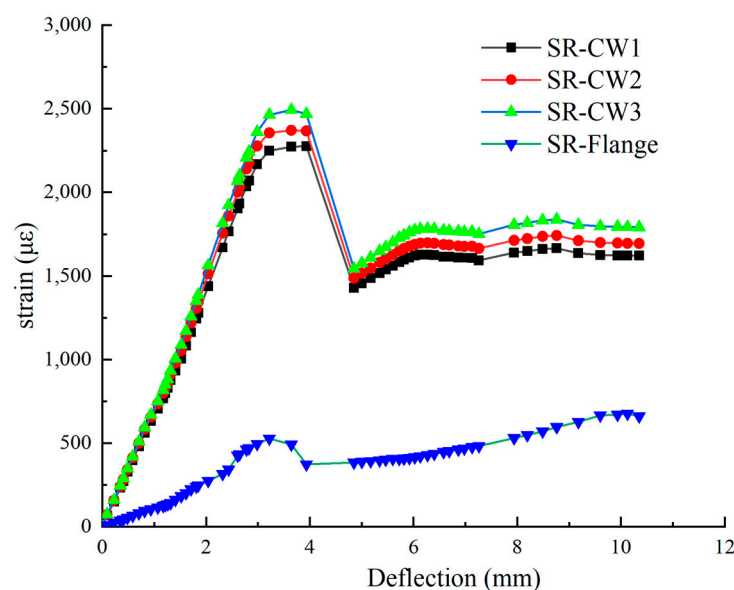
3.4. Shear Strain versus Deflection Relationships

As seen from the strain–deflection curve of the specimens in Figure 16, the notion is named according to the component and strain rosette of the CWGCFTFs. For example,

SR-CW1 means the strain rosette was pasted on the CW from channel 1 to channel 3. SR-CW2 means the strain rosette was pasted on the CW from channel 4 to channel 5. SR-CW3 means the strain rosette was pasted on the CW from channel 7 to channel 9. SR-Flange means the average value of two strain rosettes pasted on both of the lateral sides of the tube from channel 10 to channel 15. For the specimens SP-360-2-10 (Figure 16a) and SP-360-2-30 (Figure 16b), the ultimate strain is less than the shear yield strain of the CWs along the whole loading procedure. This means the shear buckling failure occurred before the shear yield of the web. However, for the specimens SP-360-3-30 (Figure 16c) and SP-180-2-30 (Figure 16d), the ultimate strain is far larger than the shear yield strain of the CWs. This indicates that the shear buckling failure occurred after the shear yield of the web. The conclusion is different from that calculated in Table 4, which could be caused by the effect of the geometrical imperfection and nonuniformity of the CW. It is more accurate to use the analysis based on the real shear strain–deflection curve of the CWGCFTFs to judge whether the CW yields in the case where the CW experiences shear buckling failure.

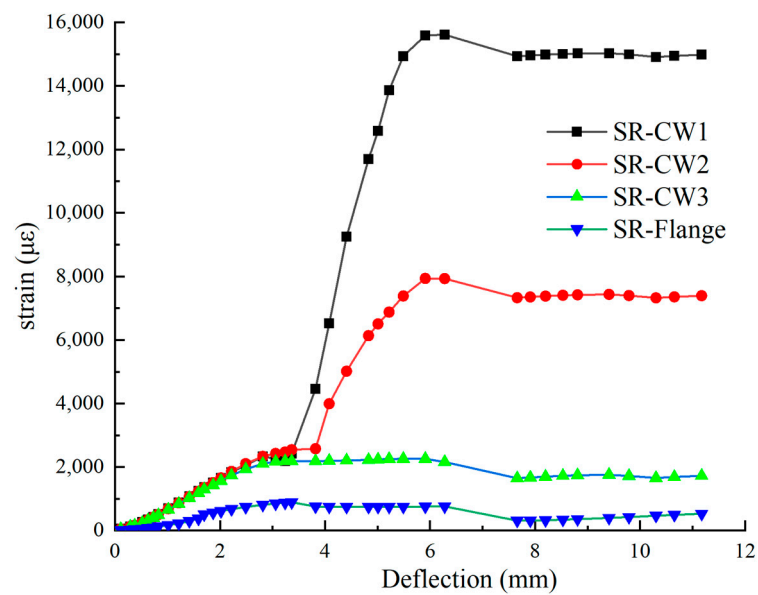


(a) Strain–deflection curve of SP-360-2-10.

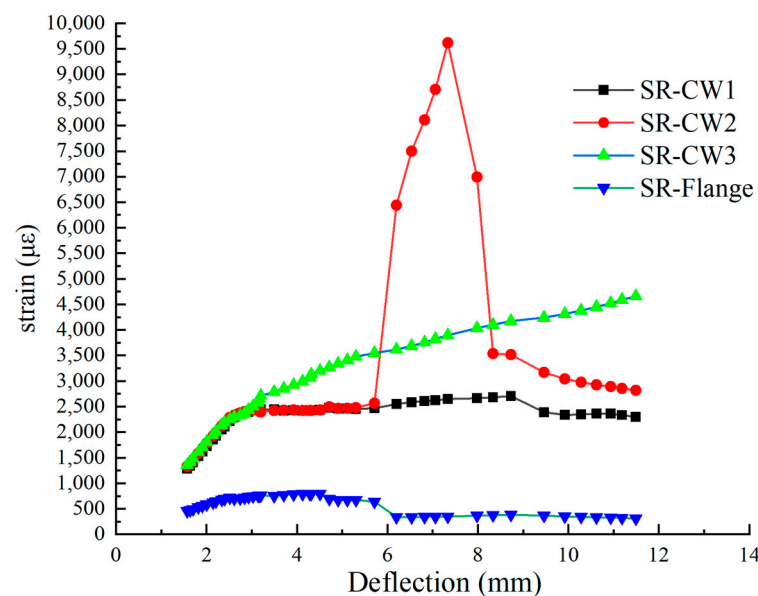


(b) Strain–deflection curve of SP-360-2-30.

Figure 16. Cont.



(c) Strain–deflection curve of SP-360-3-30.



(d) Strain–deflection curve of SP-180-2-30.

Figure 16. Strain–deflection relation of the tests.

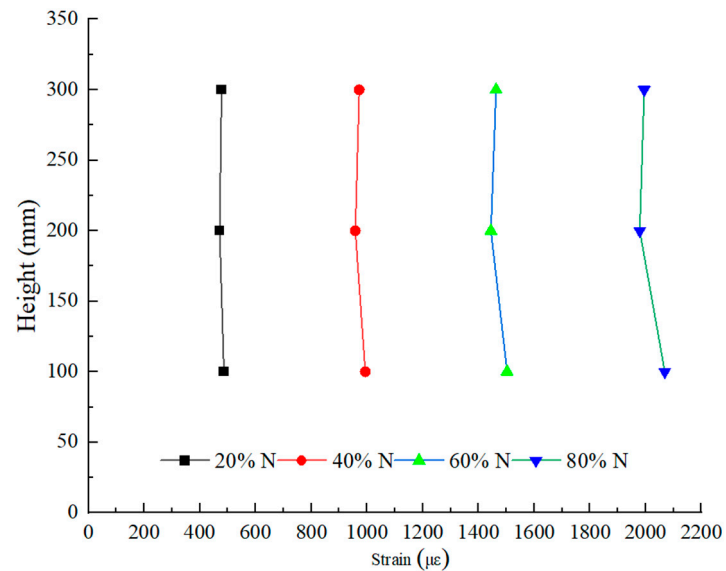
3.5. Shear Strain Distribution of the CW

The measured values of the strain rosettes of ε_0 (vertical strain), ε_{45} (inclined strain), and ε_{90} (horizontal strain) were used to calculate the shear strain of the CW (γ) by Equation (3), referring to Wang et al. [36]. The shear yield strains of the steel plates with different thicknesses were calculated by determining the yield shear strength τ_y , calculated by Equation (1) and dividing the shear elastic modulus (G) from Equation (4). The measured parameter values of the coupon tests refer to Table 3. To ensure that the shear distribution of the CWs is in the elastic stage, loads (N) at 20%, 40%, 60%, and 80% of the shear yield strains were taken. Figure 17 shows the values of the shear strain ($\mu\varepsilon$) at different positions along the height of the CWs. It is noticed and verified from the test results that the shear

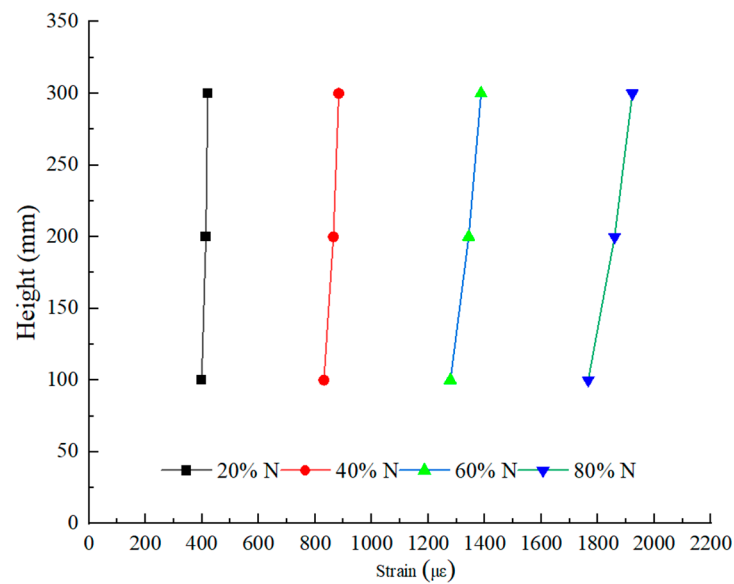
strain of the CW is distributed uniformly and the corresponding shear also has a uniform distribution in the elastic stage:

$$\gamma = \varepsilon_0 - 2\varepsilon_{45} + \varepsilon_{90} \quad (3)$$

$$G = \frac{E}{2(1+\nu)} \quad (4)$$

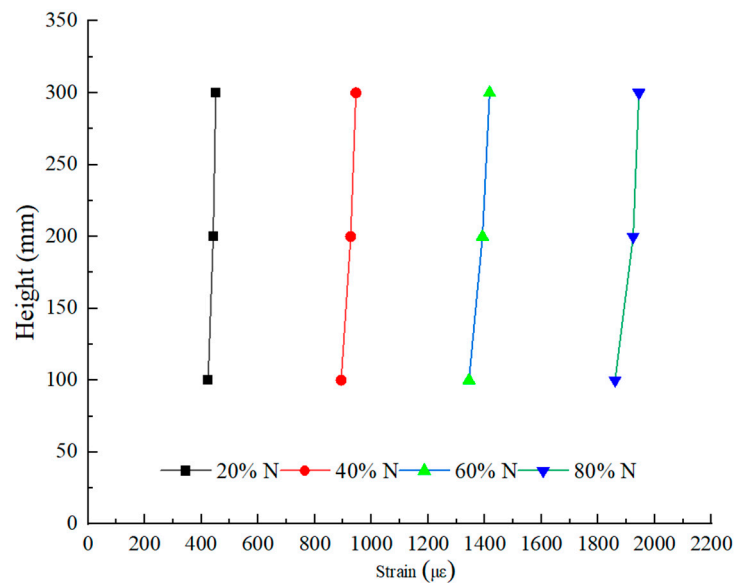


(a) CW strain distribution of SP-360-2-10.

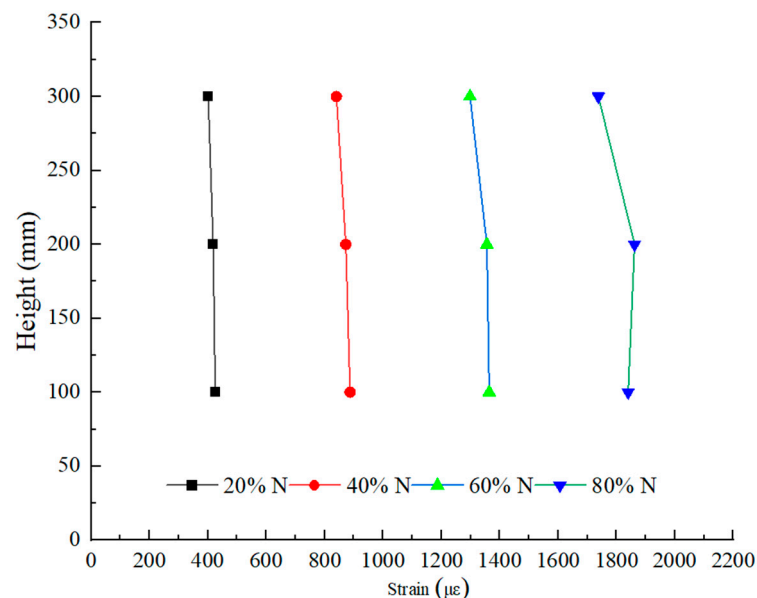


(b) CW strain distribution of SP-360-2-30.

Figure 17. Cont.



(c) CW strain distribution of SP-180-2-30.



(d) CW strain distribution of SP-360-3-30.

Figure 17. CW shear strains of test specimens.

3.6. Theoretical Derivation of the Shear Distribution and Strength Equation

In this paper, the shear ratio of the compression tubular flange to the entire cross-section is derived herein, and the shear yield strength of the CWGCFTFs is proposed. The corresponding assumptions are the following:

1. The concrete-filled tubular top flange and the bottom plate flange conform to the plane section assumption, respectively.
2. The normal stress is not generated on the CW, and it is all generated on both flanges. A small rotation appears along the length of the CWGCFTFs between the concrete-filled tubular top flange and the bottom plate flange, as shown in Figure 18.
3. The connection between the inner surface of the tube and the outer side of the filled concrete is considered a tie in the elastic stage.

where y is taken as the distance (y_o) from the neutral line of the CWGCFTFs to the neutral line of the bottom flange. Equation (7) can be modified into Equation (8):

$$\tau = \frac{V_y y_o b_{ft} t_{f2}}{I_t \times t_w} \quad (8)$$

An equivalent area is defined as the following Equation (9):

$$A_e = \frac{I_t t_w}{y_o b_{ft} t_{f2}} \quad (9)$$

The entire shear force at the flat fold cross-section of the CWGCFTFs is expressed as Equation (10):

$$V_y = \tau A_e \quad (10)$$

For the inclined fold of the CWGCFTFs, an infinitesimal part containing the bottom plate flange and the segment below the neutral line part of the CW of the CWGCFTFs is also used to analyze the axial force along the length of the girder, as shown in Figure 19b. An equation of the equilibrium can be established as Equation (11):

$$\tau \times t_w \times \cos \theta \times \frac{d_l}{\cos \theta} = \int_{A_{f2}} d_\sigma d_A \quad (11)$$

Equations (6)–(9) can be used to calculate the derived shear stress in the infinitesimal part of the inclined fold of the CWGCFTFs. The entire shear force at the inclined fold of the CWGCFTFs can be modified as per the following Equation (10). The distance from the neutral line of the CWGCFTFs to the neutral line of the bottom flange can be calculated by Equation (12):

$$y_o = h_b + t_{f2}/2 \quad (12)$$

The shear force generated on the CW can be calculated by taking the shear stress from Equation (8) and multiplying the area of the CW, as shown in Equation (13):

$$V_w = \tau A_w = \frac{V_y y_o b_{ft} t_{f2}}{I_t t_w} h_w t_w = \frac{V_y b_{ft} t_{f2}}{I_t t_w} h_w t_w (h_b + t_{f2}/2) \quad (13)$$

The shear ratio of the CW to the whole corresponding cross-section can be calculated by the following Equation (14):

$$\frac{V_w}{V_y} = \frac{b_{ft} t_{f2} h_w (h_b + t_{f2}/2)}{I_t} \quad (14)$$

The entire inertia moment of the CWGCFTFs is made of those from the sum of the section of the steel tubular flange, filled concrete, and the bottom flange on the neutral axis of the whole section, as shown in Equation (15):

$$I_t = I_{tf} + I_{bf} + I_{eqc} \quad (15)$$

where the inertia moment of the steel tubular flange and the bottom flange on the neutral axis of the whole section can be calculated by Equations (16) and (17):

$$I_{tf} = \frac{1}{12} \left[b_{ft} h_f^3 - (b_{ft} - 2t_{f1})(h_f - 2t_{f1})^3 \right] + 2t_{f1} (b_{ft} - 2t_{f1} + h_f) \left(h_t + \frac{h_f}{2} \right)^2 \quad (16)$$

$$I_{bf} = \frac{1}{12} b_{fb} t_{f2}^3 + b_{fb} t_{f2} \left(h_b + \frac{t_{f2}}{2} \right)^2 \quad (17)$$

It is mentioned that the inertia moment of the filled concrete needs to be modified to that of the steel considering the equivalent stiffness (I_{eqc}) by Equation (18):

$$I_c E_c = I_{eqc} E_s \quad (18)$$

And, the equivalent stiffness of the concrete can be calculated by the following Equation (19):

$$I_{eqc} = \left[\frac{1}{12} (b_{ft} - 2t_{f1})(h_f - 2t_{f1})^3 + (b_{ft} - 2t_{f1})(h_f - 2t_{f1}) \left(h_t + \frac{h_f}{2} \right)^2 \right] \times E_c / E_s \quad (19)$$

The CW of the CWGCFTFs does not generate normal stress while the sum of the axial force is always equal to zero. Thus, considering the rotation of both flanges of the CWGCFTFs, the summation of the axial force along the length of the girder equals zero. An equation set (Equations (20) and (21)) is used to calculate the position of the neutral line of the CWGCFTFs:

$$h_t + h_b = h_w \quad (20)$$

$$N_{tf} + N_{bf} = 0 \quad (21)$$

θ is the rotation angle of both flanges along the length of the CWGCFTFs, as shown in Figure 18. It is positioned at the neutral line of both flanges, where it does not need to consider the rotation of each flange. The compressive force and the tensile force can be calculated by Equations (22) and (23), where the notation “−” means compression and the notation “+” means tension:

$$N_{tf} = -(E_c A_c + E_s A_s)_{tf} \left(h_t + \frac{h_f}{2} \right) \theta' \quad (22)$$

$$N_{bf} = (E_s A_s)_{bf} \left(h_b + \frac{t_{f2}}{2} \right) \theta' \quad (23)$$

Thus, both the distance from the neutral line of the CWGCFTFs to the bottom surface of the top flange (h_t) and to the top surface of the bottom flange (h_b) can be calculated, as shown in Equations (24) and (25):

$$h_t = \frac{(E_s A_s)_{bf} \left(h_w + \frac{t_{f2}}{2} \right) - (E_c A_c + E_s A_s)_{tf} \frac{h_f}{2}}{(E_c A_c + E_s A_s)_{tf} + (E_s A_s)_{bf}} \quad (24)$$

$$h_b = \frac{(E_c A_c + E_s A_s)_{tf} \left(h_w + \frac{h_f}{2} \right) - (E_s A_s)_{bf} \frac{t_{f2}}{2}}{(E_c A_c + E_s A_s)_{tf} + (E_s A_s)_{bf}} \quad (25)$$

The shear yield strength equation of the CWGCFTFs can be obtained from Equation (26). f_v^w is the shear yield strength of the CW, as the CW yields before the concrete-filled tubular flange:

$$\tau = \frac{V_y}{A_e} \leq f_v^w = \frac{f_y}{\sqrt{3}} \quad (26)$$

3.7. Verification of the Theoretical Derived Equation

To analyze the shear force ratio generated on the tubular flange and the whole cross-section of the CWGCFTFs, the shear force of the CW (V_w) and tube (V_{Tu}) were computed, respectively. V_w was calculated by determining the cross-sectional area of the CW (A_w) and multiplying the shear stress (τ_w) using Equations (27) and (28). The CW shear stress (τ_w) was computed by determining the shear CW strain (γ_w) and multiplying the CW shear modulus (G_w), where the CW shear modulus (G_w) can be calculated by Equation (29). The steel tube shear force (V_{Tu}) was computed by determining the shear stress (τ_{Tu}) and multiplying both lateral sides of the tube (A_{Tu}) using Equations (30) and (31). The shear stress of the steel tube (τ_{Tu}) was computed by determining the tube shear strain (γ_{Tu}) and multiplying the shear modulus of the steel tube (G_{Tu}), where the shear modulus (G_{Tu})

can be calculated by Equation (32). The underlined value is an approximate value since the calculated shear ratio of the steel tube to the whole cross-section is larger than the compression tubular flange, with an error of no more than 4%:

$$V_w = \tau_w \times A_w \quad (27)$$

$$\tau_w = \gamma_w \times G_w \quad (28)$$

$$G_w = \frac{b+a}{b+c} \times \frac{E_w}{2(1+\nu)} \quad (29)$$

$$V_{Tu} = \tau_{Tu} \times A_{Tu} \quad (30)$$

$$\tau_{Tu} = \gamma_{Tu} \times G \quad (31)$$

$$G_{Tu} = \frac{E_{Tu}}{2(1+\nu)} \quad (32)$$

The measured dimensions of the four specimens and the elastic modulus of steel are taken to calculate the theoretical shear ratio of the CW to the entire cross-section. The elastic modulus of concrete for C10 and C30 are 22,000 MPa and 30,000 MPa, respectively, according to the code [34]. It can be calculated by the above-derived equations that the shear force of the CW to the whole cross-section is 85% for the specimen SP-360-2-10 from Equation (14). While all the shear forces of the CW to the whole cross-section are 85%, for the specimens SP-360-2-30, SP-180-2-30, and SP-360-3-30, they are 86%. Comparing the full-scale experimental shear test results, the ratio of the measured shear of the CW to the whole cross-section is unstable in the beginning. When the load reaches 80% of the ultimate shear yield load, the measured shear force of the CW to the whole cross-section is 85%, calculated by the unit minus the ratio of the flange to the whole cross-section as the number in the bracket in Table 5, which is almost the same as the derived shear ratio of the CW to the entire section. For the other specimens, the measured shear ratio of the CW to the entire cross-section is about 82–83% at 80% of the ultimate shear yield load (f), whose error compared with the derived shear ratio of the CW to the whole cross-section is about 3–4%. The relatively large error between the derived and measured values of the shear ratio of the CW to the whole cross-section could be caused by the geometrical imperfection of the CWs. Therefore, the shear ratio equation generated on the CW to that of the whole cross-section is verified.

Table 5. Shear at different load levels of the CWGCFTFs (kN).

Specimen No.	Element	Load Level			
		20% f	40% f	60% f	80% f
SP-360-2-10	Tube (V_{Tu})	6.7 (23%)	12.1 (21%)	17.2 (20%)	21.3 (18%)
	CW (V_w)	23.9	48.5	73.1	99.6
	CW (V_w)	23.5	47.8	72.1	98.8
	CW (V_w)	24.3	49.6	75.0	103.3
	Flange ($V_{Tu} + V_{Co}$)	<u>5.6 (19%)</u>	<u>10.3 (18%)</u>	<u>14.6 (17%)</u>	<u>17.4 (15%)</u>
SP-360-2-30	Tube (V_{Tu})	2.6 (9%)	5.3 (9%)	9.8 (11%)	13.6 (12%)
	CW (V_w)	22.2	46.6	73.1	101.4
	CW (V_w)	21.8	45.6	70.8	98.1
	CW (V_w)	21.0	43.8	67.4	93.2
	Flange ($V_{Tu} + V_{Co}$)	7.3 (25.3%)	13.6 (23.1%)	17.5 (19.9%)	20.5 (17.3%)
SP-180-2-30	Tube (V_{Tu})	4.8 (16%)	10.6 (17%)	18.8 (21%)	24.7 (20%)
	CW (V_w)	23.8	49.9	74.8	102.6
	CW (V_w)	23.3	48.9	73.4	101.5
	CW (V_w)	22.3	47.1	70.9	98.1
	Flange ($V_{Tu} + V_{Co}$)	6.9 (23%)	12.4 (20%)	17.0 (19%)	<u>20.3 (17%)</u>
SP-360-3-30	Tube (V_{Tu})	3.8 (9%)	8.8 (10%)	20.2 (16%)	26.6 (16%)
	CW (V_w)	30.3	63.5	98.1	131.4
	CW (V_w)	31.6	66.0	102.5	140.8
	CW (V_w)	32.2	67.1	103.1	139.1
	Flange ($V_{Tu} + V_{Co}$)	10.6 (25%)	19.5 (23%)	26.3 (21 %)	30.4 (18 %)

4. Conclusions

This paper researched the shear behavior of CWGCFTFs, namely corrugated web girders with concrete-filled top compression tubular flanges and bottom tension plate flanges. Four full-scale small-size shear tests of the CWGCFTFs were conducted, and the corresponding theoretical equations of the shear yield strength were derived. Based on the current research, some conclusions can be drawn:

1. The failure mode of the current shear test of the CWGCFTFs shows that two roller supports at both ends cannot express the simple support for the shear behavior analysis accurately, and a novel simple support device with jacks to constrain the displacement of the specimen along its length direction is proposed.
2. The rule of shear distributed uniformly along the height of the CW of the CWGCFTFs is verified by the current shear tests. The test results indicate that the failure modes of the CWGCFTFs under shear are the shear buckling failure of the CW accompanied by compression buckling at the top surface of the steel compression tubular flange.
3. A theoretical equation for the shear force of the CW to that of the whole cross-section is derived, and a shear yield strength equation of the CWGCFTF is proposed and verified by comparisons with the test results.

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References

1. Hassanein, M.; Elkawas, A.; El Hadidy, A.; Elchalakani, M. Shear Analysis and Design of High-strength Steel Corrugated Web Girders for Bridge Design. *Eng. Struct.* **2017**, *146*, 18–33. [\[CrossRef\]](#)
2. Deng, H.; Shao, Y.B.; Hassanein, M.F. Experimental Shear Testing of Small-scale Corrugated Web Girders Applied in Conventional Buildings. *J. Constr. Steel Res.* **2022**, *189*, 107086. [\[CrossRef\]](#)
3. Leblouba, M.; Tabsh, S.W. Reliability-based Shear Design of Corrugated Web Steel Beams for AISC 360 Specification and CSA-S16 Standard. *Eng. Struct.* **2020**, *215*, 110617. [\[CrossRef\]](#)
4. Hassan, H. Abbas. Analysis and Design of Corrugated Web I-girders for Bridges Using High Performance Steel. Ph.D. Thesis, Lehigh University, Bethlehem, PA, USA, 2003.
5. Wang, X. Behavior of Steel Members with Trapezoidally Corrugated Webs and Tubular Flanges under Static Loading. Ph.D. Thesis, Drexel University, Philadelphia, PA, USA, 2003.
6. Hassanein, M.F.; Kharoob, O.F. Shear Strength and Behavior of Transversely Stiffened Tubular Flange Plate Girders. *Eng. Struct.* **2010**, *32*, 2617–2630. [\[CrossRef\]](#)
7. Shao, Y.B.; Wang, Y.M. Test Study on Static Behavior of I-girder with Concrete-Filled Hollow Flange and Corrugated Web under Concentrated Load at Mid-Span. *Eng. Struct.* **2017**, *130*, 124–141. [\[CrossRef\]](#)
8. Wang, S.; Zhang, Y.; Luo, T.; Liu, Y. Elastic critical shear buckling stress of large-scale corrugated steel web applied in bridge girders. *Eng. Struct.* **2021**, *244*, 112757. [\[CrossRef\]](#)
9. Ling, H.; Hikosaka, H.; Komine, K. Simulation of Accordion Effect in Corrugated Steel Web with Concrete Flanges. *Comput. Struct.* **2004**, *82*, 2061–2069.
10. Yue, J.O.; Lee, F.D.; Kim, K.S. Accordion Effect of Prestressed Steel Beams with Corrugated Webs. *Thin-Walled Struct.* **2012**, *57*, 49–61.
11. Zhou, M.; Shang, X.; Hassanein, M.F.; Zhou, L. The Differences in the Mechanical Performance of Prismatic and Non-prismatic Beams with Corrugated Steel Webs: A Comparative Research. *Thin-Walled Struct.* **2019**, *141*, 402–410. [\[CrossRef\]](#)

12. Yi, J.; Gil, H.; Youm, K.; Lee, H. Interactive Shear Buckling Behavior of Trapezoidally Corrugated Steel Webs. *Eng. Struct.* **2008**, *30*, 1659–1666. [\[CrossRef\]](#)
13. Sause, R.; Thomas, N.; Braxtan, T.N. Shear Strength of Trapezoidal Corrugated Steel Webs. *J. Constr. Steel Res.* **2011**, *67*, 223–236. [\[CrossRef\]](#)
14. Nie, J.-G.; Zhu, L.; Tao, M.-X.; Tang, L. Shear Strength of Trapezoidal Corrugated Steel Webs. *J. Constr. Steel Res.* **2013**, *85*, 105–115. [\[CrossRef\]](#)
15. Hassanein, M.F.; Kharoob, O.F. Behavior of Bridge Girders with Corrugated Webs: (I) Real Boundary Conditions at the Juncture of the Web and Flanges. *Eng. Struct.* **2013**, *57*, 554–564. [\[CrossRef\]](#)
16. Hassanein, M.F.; Kharoob, O.F. Behavior of Bridge Girders with Corrugated Webs: (II) Shear Strength and Design. *Eng. Struct.* **2013**, *57*, 544–553. [\[CrossRef\]](#)
17. Leblouba, M.; Barakat, S.; Al-Saadon, Z. Shear Behavior of Corrugated Web Panels and Sensitivity Analysis. *J. Constr. Steel Res.* **2018**, *151*, 94–107. [\[CrossRef\]](#)
18. Amani, M.; Al-Emrani, M.; Flansbjerg, M. Shear Behavior of Stainless Steel Girders with Corrugated Webs. *J. Constr. Steel Res.* **2023**, *210*, 108086. [\[CrossRef\]](#)
19. Lee, H.-D.; Lee, S.-H.; Shin, K.-J.; Lee, J.-S. Shear Strength Evaluation of Steel Beams with Partially Corrugated Web. *J. Constr. Steel Res.* **2023**, *211*, 108179. [\[CrossRef\]](#)
20. Wang, S.; Zhang, K.; Zhang, Y.; Liu, Y. Shear Failure Mechanism of Local Buckling-Dominated Large-Scale Corrugated Steel Web. *Thin-Walled Struct.* **2023**, *182*, 110279. [\[CrossRef\]](#)
21. Hu, B.; Wu, Y.F. Effect of Shear Span-to-Depth Ratio on Shear Strength Components of RC Beams. *Eng. Struct.* **2018**, *168*, 770–783. [\[CrossRef\]](#)
22. Wen, Z.; Wei, X.; Xiao, L.; He, K. Experimental Evaluation of the Shear Buckling Behaviors of Corrugated Webs with Artificial Corrosion Pits. *Eng. Struct.* **2019**, *141*, 251–259. [\[CrossRef\]](#)
23. Zhang, B.; Yu, J.; Chen, W.; Wang, H.; Xu, J. Stress States and Shear Failure Mechanisms of Girders with Corrugated Steel Webs. *Thin-Walled Struct.* **2020**, *157*, 106858. [\[CrossRef\]](#)
24. Wu, Y.; Du, X.; Yuan, H.; Zhou, M. Shear Behaviour and Design of Cold-Formed Steel Box Girders with Tubular Flanges and Sinusoidal Corrugated Webs. *Thin-Walled Struct.* **2022**, *174*, 109066. [\[CrossRef\]](#)
25. Deng, L.; Shao, Y.; Jiang, D.; Hassanein, M.F.; He, J. Elastic Analysis of I-girders with Tubular Flange and Corrugated Web. *J. Constr. Steel Res.* **2024**, *213*, 108407. [\[CrossRef\]](#)
26. Shao, Y.B.; Wang, Y.M. Experimental Study on Shear Behavior of I-girder with Concrete-Filled Tubular Flange and Corrugated Web. *Steel Compos. Struct.* **2016**, *22*, 1465–1486. [\[CrossRef\]](#)
27. Wang, Y.M.; Shao, Y.B. Stress Analysis of a New Steel-Concrete Composite I-Girder. *Steel Compos. Struct.* **2018**, *28*, 51–61.
28. Wang, Y.M.; Shao, Y.B.; Chen, C.; Katwal, U. Prediction of flexural and shear yielding strength of short span I-girders with concrete-filled tubular flanges and corrugated web—I: Experimental test. *Thin-Walled Struct.* **2020**, *148*, 106592. [\[CrossRef\]](#)
29. Wang, Y.M.; Shao, Y.B.; Chen, C.; Katwal, U. Prediction of flexural and shear yielding strength of short span I-girders with concrete-filled tubular flanges and corrugated web—II: Numerical simulation and theoretical analysis. *Thin-Walled Struct.* **2020**, *148*, 106593. [\[CrossRef\]](#)
30. Deng, H.; Shao, Y.B.; Hassanein, M.F. Experimental Shear Testing of Corrugated Web Girders with Compression Tubular Flanges Applied in Conventional Buildings. *Thin-Walled Struct.* **2022**, *179*, 109557. [\[CrossRef\]](#)
31. Deng, H.; Hassanein, M.; Shao, Y.-B.; He, J. Shear Mechanism and Design of Small-Scale Tubular Flange Corrugated Web Girders. *J. Constr. Steel Res.* **2022**, *197*, 107501. [\[CrossRef\]](#)
32. Deng, H.; Shao, Y.B.; Hassanein, M.F. Shear Strength of Corrugated Web Girders with Compression Tubular Flanges. *Structures* **2024**, *62*, 106245. [\[CrossRef\]](#)
33. Zhang, Z.B.; Liu, H.; Hou, S.L.; Chen, F. Experimental Study on Mechanical Properties of Superfine Sand and Coal Gangue Concrete. *Coal Sci. Technol.* **2022**, *50*, 57–66. (In Chinese)
34. GB 50010-2010; Code for Design of Concrete Structures. Standardization Administration of China: Beijing, China, 2015. (In Chinese)
35. GB/T 228.1-2010; Metallic Materials-Tensile Testing-Part 1: Method of Test at Room Temperature. Standardization Administration of China: Beijing, China, 2010. (In Chinese)
36. Wang, S.H.; He, J.; Liu, Y.Q. Shear Behavior of Steel I-Girder with Stiffened Corrugated Web, Part I: Experimental Study. *Thin-Walled Struct.* **2019**, *140*, 248–262. [\[CrossRef\]](#)
37. Wu, W.Q.; Ye, J.S.; Wan, S.; Hu, C. Quasi Plane Assumption and Its Application in Steel-concrete Composite Box Girders with Corrugated Steel Webs. *Eng. Mech.* **2005**, *22*, 177–181. (In Chinese)

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