



Steel-concrete composite corrugated web girders – Experimental study

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ABSTRACT

Corrugated web girders are widely used in buildings and bridges. This paper presents an experimental research program focused on prefabricated steel-concrete composite bridge girders with corrugated steel web. Web is the corrugated steel plate, bottom flange fabricated from steel, and upper flange is a reinforced concrete. The transfer of shear flow between the CW (corrugated web) and the concrete flange is facilitated by two mechanisms: (i) the embedding of the steel corrugated web into the reinforced concrete flange and (ii) the use of transverse rebars that pass through the web openings in the embedded web sections. Prior push-out tests conducted by the authors [1] suggest that this shear connection is likely to ensure complete shear flow transfer. The aim of this research is to investigate the bending and shear behavior, as well as the interaction between bending and shear. The pure bending resistance is primarily influenced by the accordion effect, rendering the contribution of the CW negligible. Consequently, the focus of the study is to examine the shear contribution of the upper reinforced concrete flange to the overall shear capacity of the section under varying bending moments. In addition to assessing (i) resistance measurements, the study evaluates (ii) initial stiffness, (iii) failure mechanisms, (iv) collapse modes, and (v) post-buckling strength. Available experimental tests in the literature are quite limited. This paper presents laboratory tests conducted on five specimens subjected to eight different load scenarios. Based on the experimental results, a preliminary design proposal is developed for the bending-shear interaction resistance.

1. Introduction and literature review

Composite steel-concrete beams exhibit significantly enhanced longitudinal and torsional stiffness compared to conventional steel girders, making them particularly suited for bridge applications. The first hybrid corrugated web (CW) bridge was constructed in France in 1986, known as the Cognac Bridge, followed by the Maupré Viaduct in 1987. Since then, numerous composite bridges with corrugated web girders have been erected across Asia, particularly in Japan, China, and South Korea. In Hungary, two bridges utilizing this technology have been built: the Móra Ferenc Bridge in 2011 and the Paks-Kalocsa Bridge in 2024.

As mentioned above prior push out tests are executed by the authors [1]. The pure bending resistance is primarily influenced by the accordion effect, rendering the contribution of the CW negligible [2].

A considerable research has explored the shear behavior of corrugated web girders, both with pure steel flanges and tubular flanges. The international literature contains numerous papers addressing the pure shear buckling resistance of corrugated webs, many of which provide design proposals [3–17]. EN1993-1-5 Annex D [18] offers design guidelines only for pure steel corrugated webs. Currently, there is no

established proposal in the standards or literature for the integrated shear resistance of steel-concrete composite beams with corrugated webs, although designers and researchers acknowledge the significant shear contribution of the concrete flanges. Furthermore, the bending-shear interaction resistance of steel-concrete composite beams with corrugated webs has not been thoroughly investigated.

In 2013, Nie et al. [8] conducted an experimental research program to evaluate the shear strength of corrugated web girders, testing eight samples with two flange widths: 150 mm and 200 mm. Measuring devices were placed along the web height to collect data for analysis. The results indicated a uniform stress distribution along the height of the girder, and the formula developed by Johnson and Cafolla [19] provided accurate predictions for the shear modulus of the corrugated web girders. In 2016, Y.B. Shao et al. [20] focused on tubular upper flanges, comparing ordinary I-beams with corrugated web girders. Their research primarily aimed to investigate material failure mechanisms. Findings indicated that the corrugated web exhibited greater buckling resistance when subjected to shear forces. Additionally, the use of concrete-filled tubular flanges was shown to enhance resistance to local buckling capacity and distortion. Shao and Wang [21] conducted

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laboratory tests in 2017 to evaluate the performance of an I-girder with a corrugated web and a concrete-filled tubular flange. Their study revealed that the concrete-filled tubular flange provided greater stiffness than a flat steel plate. During testing, when significant pressure was applied to the center of the girder, the normal I-girder experienced bending type failure with buckling of the plates. In contrast, the composite design with the corrugated web and concrete-filled tubular flange exhibited bending without buckling.

In 2017, Leblouba et al. [22] carried out an experimental investigation involving 12 beams subjected to concentrated shear forces at mid-span. The beams, with a thickness of 1.2 mm, were tested across three different web depths under simple support conditions to ensure reliable results. Their findings highlighted the crucial role that corrugation geometry plays in shear resistance, with a 45-degree angle showing superior performance compared to shallower angles. In 2018, Leblouba et al. [23] evaluated the shear behavior of corrugated web girders with varying flange resistances. Nine specimens underwent three-point loading tests, with high-strength steel (HSS) utilized for the flanges. The results indicated that shear buckling occurred prior to yielding in six of the specimens, and all failures were due to shear buckling, with the post-failure web exhibiting 60 % of its original shear strength. Shao et al. [24] conducted an experimental investigation in 2020 comparing corrugated and flat web girders with upper tubular flanges filled with concrete. This study aimed to assess performance under short-span conditions subjected to shear forces similar to three-point tests. The results suggested that a well-designed system could exhibit flexural yielding in the bottom flange under specific conditions. In 2022, Hassanein et al. [25] tested three small-scale specimens under shear, with web thicknesses ranging from 2 mm to 3 mm and heights of 380 mm and 420 mm. The experiments, supported by hinged and roller conditions, indicated uniform shear stress distribution along the web height. Increasing web thickness showed a positive impact on shear strength and ultimate behavior compared to increasing web height. Hassanein et al. [26] explored trapezoidal web girders with tubular flanges, testing three full-scale specimens to assess the effects of web thickness and corrugation wavelength on shear performance. Results revealed that increased slenderness ratios shifted the failure mode from the corrugated web to the web itself.

El-Metwally [27] investigated prestressed composite girders with corrugated steel webs in 1998. In this research program five full-scale hybrid girders were under investigation. Main aims of the study were to theoretically investigate the shear capacity of the corrugated web girders and experimentally investigate the flexural and shear behavior. In 2015 Jiang et al. [28] studied prestressed concrete bridges with corrugated web girders. They investigated the shear resistance of embedded corrugated web in concrete flanges, which could have significant shear carrying capacity depending on the shear connector type. In 2019 Górecki and Sledziwski [29] executed experimental research program on sinusoidally corrugated web girders with upper concrete flange. Beams were loaded with four-point bending in the midspan.

In 2024, Hao Deng et al. [30] conducted tests on four full-scale specimens to investigate shear failure in corrugated web girders featuring concrete-filled top compression tubular flanges and bottom tension flanges. The testing involved applying mid-span loads. The specimens were supported by rolled steel beams, but the results indicated that the roller supports failed to accurately simulate the behavior of simple supports under shear conditions. Also in 2024, Yang and Bi [31] performed both laboratory tests and numerical investigations on a steel-concrete composite girder. The girders were constructed with a steel bottom flange and web, while the upper flange was secured to the upper steel flange using shear studs (connectors). The experiments utilized various shear span ratios to assess the failure modes of the specimens. The findings indicated a transition from shear-dominated failure to flexural-dominated failure, closely linked to changes in the shear span ratio. Specifically, a decrease in the shear span ratio led to the prevalence of shear-dominated failure, whereas an increase in this ratio

resulted in bending-type failure. Moreover, the study revealed that the upper concrete flange contributes approximately 15 % to 35 % of the total shear resistance, confirming the accuracy and precision of existing design formulas of the international literature for calculating the shear contribution coming from the concrete flange. EN1993-1-5 Annex D separately considers two buckling modes such as local and global buckling in case of shear resistance calculation of corrugated web girders. Local buckling can be considered according to the conventional isotropic plate buckling theory [18] shown by Eq. (1).

$$\tau_{cr,L}^{el} = k_L * \frac{\pi^2 * E}{12 * (1 - \nu^2) * \left(\frac{a_{max}}{t_w}\right)^2} \quad (1)$$

where k_L equals 5.34 (simply supported edges) to 8.98 (clamped edges), $\nu = 0.3$ is the Poisson-ratio. Eq. (1) can be simplified for simply supported edges according to Eq. (2):

$$\tau_{cr,L}^{el} = 4.83 * E * \left(\frac{t_w}{a_{max}}\right)^2 \quad (2)$$

Considering global shear buckling when all folds are involved in buckling Easley and McFarland [9] proposed a formula based on orthotropic plate buckling theory in form of Eq. (3):

$$\tau_{cr,G}^{el} = k_G * \frac{D_z^{0.75} * D_x^{0.25}}{t_w * h_w^2} \quad (3)$$

where k_G is 31.6 if the web to flange connection is simply supported, and k_G equals 59.0 for an idealized clamped connection. In Eq. (3) D_z and D_x are the bending stiffnesses of the web. These can be calculated according to Eqs. (4) and (5):

$$D_z = \frac{3a_1 + a_2 * (a_3)^2 * E * t_w}{a_1 + a_4} \quad (4)$$

$$D_x = \frac{a_1 + a_4 * t_w^3 * E}{a_1 + a_2 * 12 * (1 - \nu^2)} \quad (5)$$

where t_w is web thickness, E is Young's modulus, a_1, a_2, a_3, a_4 are geometrical parameters as shown in Fig. 1.

The design model of the EN 1993-1-5 for shear resistance calculation of corrugated web girders is given in form of Eq. (6).

$$V_{bw,Rd} = \chi_c * \frac{f_{yw}}{\gamma_{M1} * \sqrt{3}} * h_w * t_w \quad (6)$$

where χ_c is the reduction factor considering local and global shear buckling, t_w is the web thickness, h_w is the web depth, f_{yw} is the yield strength of the web plate, γ_{M1} is the partial safety factor for stability check. For local and global shear buckling the relative slenderness can be determined separately according to Eqs. (7) and (8), respectively.

$$\lambda_{c,L} = \sqrt{\frac{\tau_y}{\tau_{cr,L}^{el}}} \quad (7)$$

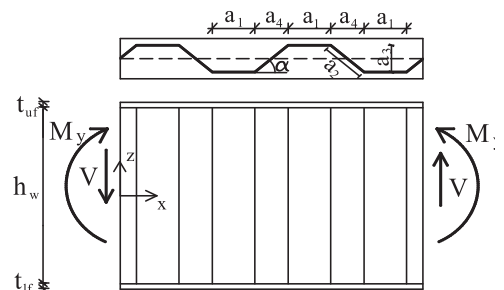


Fig. 1. Used notations for the corrugated web girder.

$$\lambda_{c,G} = \sqrt{\frac{\tau_y}{\tau_{cr,G}^{el}}} \quad (8)$$

Having the relative slenderness the reduction factors for local and global shear buckling can be computed according to Eqs. (9) and (10), respectively. It was experienced that in case of local buckling the post-buckling behavior is considerable in comparison with the global buckling where it is negligible.

$$\chi_{c,L} = \frac{1,15}{0,9 + \lambda_{c,L}} \leq 1,0 \quad (9)$$

$$\chi_{c,G} = \frac{1,5}{0,5 + \lambda_{c,G}^2} \leq 1,0 \quad (10)$$

The final general reduction factor ($\chi_{c,}$) is the minimum of the above given two reduction factors regarding local and global shear buckling according to Eq. (11).

$$\chi_c = \min(\chi_{c,L}; \chi_{c,G}) \quad (11)$$

To test and check the accuracy and applicability of the Eurocode based design models for composite corrugated web girders having concrete compression flange, a laboratory test program is designed and executed. Results are introduced in the following sections.

2. Experimental investigation

2.1. Test specimens

The main purpose of the current research is to determine the bending-shear interaction resistance of composite beams, and to understand the shear contribution of the upper concrete flange to the integrated shear resistance of the whole composite cross-section. In the tests all specimens are the same but the boundary and loading conditions are different. In the international literature only limited experimental investigations are available with upper tubular and reinforced concrete flanges for CW girders [26,32–34]. Therefore, an experimental research program is carried out at the Budapest University of Technology and Economics (BME), Department of Structural Engineering. Five large scale test specimens are investigated under three-point-bending. Girders are simply supported at the two ends and laterally supported at the load introduction places. All specimens have the same geometrical properties. The nominal width of the lower steel flange is 250×18 mm. Width of the upper concrete flange is 350 mm with thickness of 100 mm. The full height of the specimen is 560 mm. The web is embedded into the top concrete flange by 60 mm depth. Every test girder have a web thickness of 4 mm. Figs. 2 to 4 show the geometrical properties of the specimens. The corrugation angle is 45° and the inclined and parallel web fold lengths are 98 mm. Concrete cover for rebars is 15 mm in each direction. Transverse rebars go through web openings with 10 mm diameter. Longitudinal rebars have diameter of 6 mm.

The concrete grade is C40/50 according to EN 206 [35]. Test steel grade is S355. Tensile coupon tests are carried out to determine the material quality of the flange and web (steel and concrete as well).

The average yielding and tensile strength of the lower flange are 271

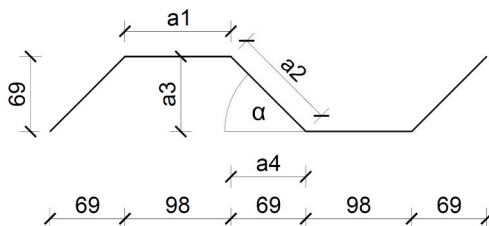


Fig. 2. Corrugation profile - web geometry.

MPa and 413 MPa, respectively. The average yield and tensile strength of the webs are 309 MPa and 393 MPa, respectively. The concrete compression strength is verified for C40/50.

2.2. Test setup and instrumentation

The experimental research program utilized two test arrangements: simply supported beams (i) without overhangs and (ii) with overhangs. Initially, each girder is tested without overhangs, as illustrated in Fig. 5. Subsequently, specimens B1 and B5 are tested again with overhangs, as shown in Fig. 6. The supports and load introduction points near the damaged regions are shifted inwards to allow for additional testing of the undamaged portions of the specimens. In total, specimen B1 underwent three tests, while specimen B5 is tested twice. Vertical stiffeners are attached to the web panel at each loading and support position to prevent local failure. Table 1 summarizes the dimensions of the test layouts for the various specimens, using the same notation as in Fig. 5 and Fig. 6. This table provides essential information on the loading positions and girder span lengths, indicating that three tests are conducted with overhangs. To further enhance stability, the specimens are laterally supported, as depicted in Fig. 7, to prevent lateral motion. Loading is applied using a hydraulic jack with a maximum capacity of 1000 kN. Vertical displacements are measured and recorded at the load introduction point and beneath the vertical supports. Fig. 4 illustrates the instrumentation of the specimens. The load is introduced through a pinned heavy steel plate positioned below the hydraulic jack. Additionally, two strain gauges are placed on the top of the bottom tension flange directly under the hydraulic jack to monitor strains. Support movements are also recorded alongside the deflection of the specimens using linear variable displacement transducers.

A test arrangement is shown in Fig. 7; this arrangement belongs to the B1/1 specimen. In this configuration the load is applied 1 m distance from the right support.

3. Evaluation of the test results

3.1. Overall test results

During the experiments, three types of failure modes are observed:

- (i) **Shear type failure mode** with shear buckling of the web panel and cracking of the upper concrete flange at the force introduction. Progressive development of the descending branch after the maximum force level. Significant residual strength after force drop.
- (ii) **Bending type failure mode** with column type failure of the upper concrete flange: sudden rebar buckling and concrete crushing type failure of the concrete flange at the top of the equilibrium path. Sudden descending branch after the maximum force level. No residual strength after force drop.
- (iii) **Bending-shear interaction failure** with progressive crushing of the upper concrete flange without shear buckling of the web panel and with shear failure type force-displacement equilibrium path. Moderate residual strength after force drop.

The failure is governed by the load introduction position. When load is introduced close to a support, (i) shear type failure is observed while the load is introduced close to the middle of the span, (ii) bending type failure has occurred.

In general, the shear-type failure shows different character in the force-displacement equilibrium path than the bending-type failure. Table 2 presents the failure modes, maximum force and maximum internal forces (bending moment and shear force) for each specimen. Table 3 collects the measured initial and hardening stiffnesses of specimens calculated from the force-displacement equilibrium path. In the presented force-displacement diagrams (Figs. 8 to 12) the green dots

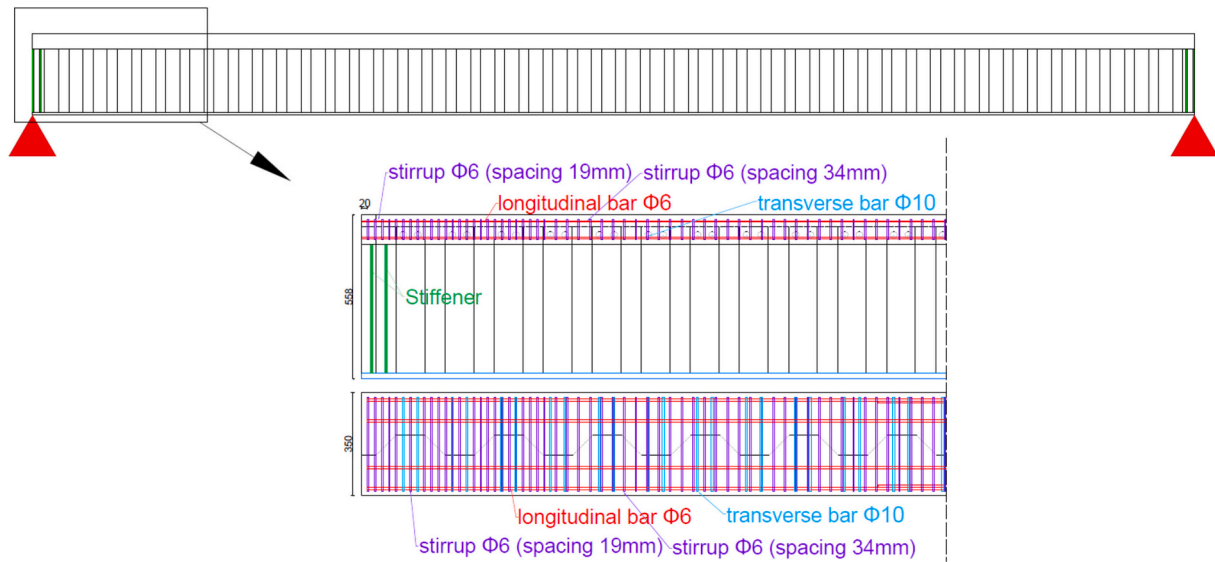


Fig. 3. Reinforcement of the beam (longitudinal view).

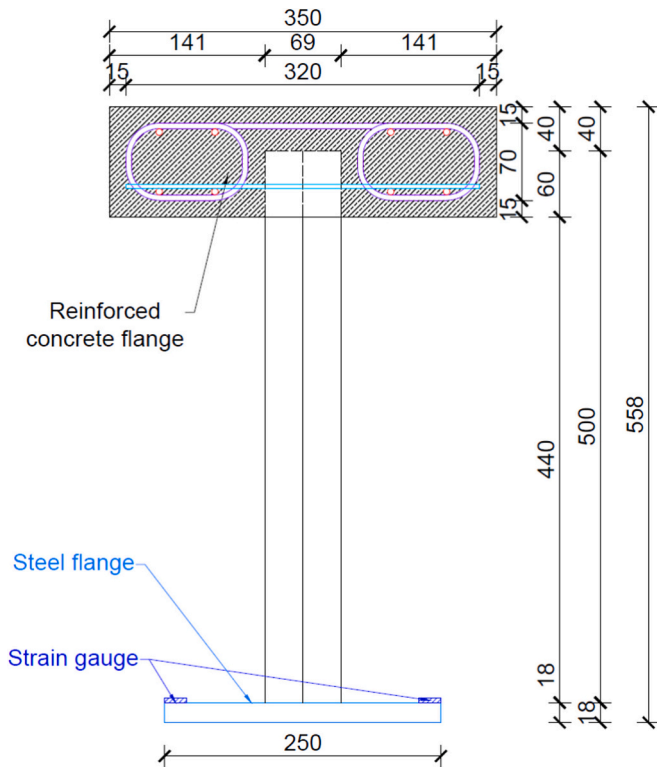


Fig. 4. Longitudinal bars and stirrups; cross-section and reinforcements.

(point “A”) represent the end of the initial phase, and the red dots (point “B”) represent the maximum force value at the end of the hardening phase. Based on the points the characteristic stiffnesses are computed. It can be observed that the closer is the force location to the end vertical support, the greater are the initial and hardening stiffnesses.

During the experimental investigation vertical displacements and the introduced force are measured. In Figs. 8 to 12 vertical displacement – force relationships are shown for all the 8 specimens. Stiffness changes during the loading procedure for each test. Initial stiffness regards to the elastic state of the specimens between points “0” and “A” in Figs. 8 to 12. After point “A” a secondary, hardening stiffness develops when at least one of the structural element (steel flange, steel web, concrete flange) individual stiffness changes due to yielding, buckling or concrete cracking/degradation. At point “B” the whole section fails by the failure of at least two structural elements of the cross-section. The failure mechanisms are different for the shear and bending type failures.

In the case of **shear type failure** (see later on in Fig. 8, Fig. 13):

- (i) Initial stiffness is governed by the elastic state of the structural elements.
- (ii) After point “A” the hardening stiffness is governed by the shear deformation of the web panel which therefore do not provide full shear transfer between the flanges. Between points “A” and “B” the stiffness is governed by the separate local bending stiffnesses of the flanges, thus mostly the separate local bending stiffnesses of the upper concrete flange; since the lower steel flange utilizes its local bending capacity right after point “A” (according to strain measurements).
- (iii) At point “B” the concrete flange reaches its normal force and bending interaction load carrying capacity and fails by dominant

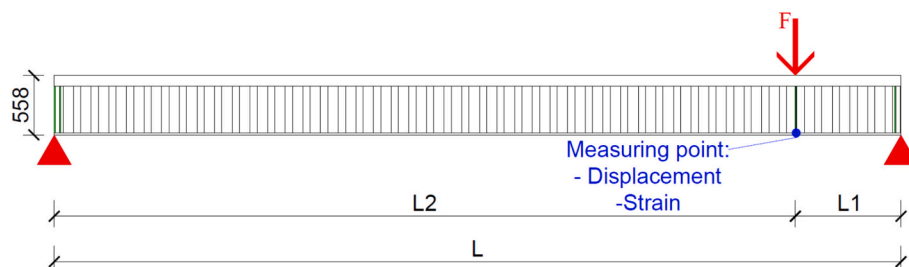


Fig. 5. Test arrangement without overhang (B1/1, B2, B3, B4, B5/1).

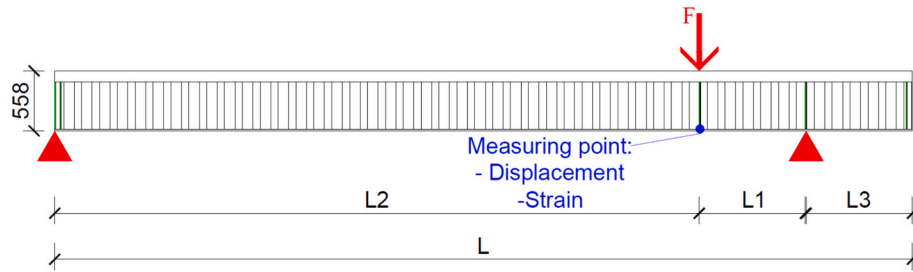


Fig. 6. Test arrangement with overhang (B1/2, B1/3, B5/2).

Table 1

Dimensions of test layouts.

LOADING		BEAM WO CANTILEVER		BEAM WITH CANTILEVER		
		L1 [m]	L2 [m]	L1 [m]	L2 [m]	L3 [m]
B1	B1/1	1.00	7.00	–	–	–
	B1/2	–	–	1.00	6.00	1.00
	B1/3	–	–	1.00	5.00	2.00
B2		4.00	4.00	–	–	–
B3		2.00	6.00	–	–	–
B4		3.00	5.00	–	–	–
B5	B5/1	1.33	6.67	–	–	–
	B5/2	–	–	1.50	5.17	1.33

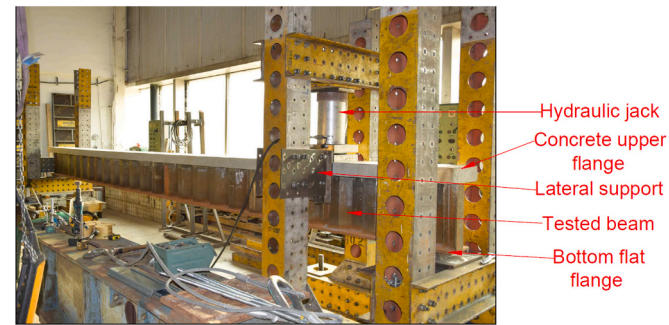


Fig. 7. Test setup (B1/1).

Table 2

Failure modes and measured ultimate loads.

LOADING	Failure mode	Forces from selfweight and load		Maximum load (point "B") [kN]
		Moment [kNm]	Shear [kN]	
B1	B1/1	380	379	428
	B1/2	384	384	443
	B1/3	408	411	495
B2	Bending	756	186	372
B3	Bending	654	325	430
B4	Bending	731	241	384
B5	B5/1	514	385	457
	B5/2	521	347	444

local bending failure (upper compression extreme fibre crushes with lower tension extreme fibre cracks).

- (iv) Notable residual strength remains after the force drop at point "B". It is attributed to the development of a stabilizing diagonal web tension zone (due to interactive shear buckling of the adjacent web folds).

In the case of **bending type failure** (see later on in Fig. 11, Fig. 19):

Table 3

Measured bending stiffness for all specimens in [kN/mm].

LOADING		Force [kN]		EI_1	EI_2	EI_2 / EI_1
		Point "A"	Point "B"			
B1	B1/1	379	428	33.2	1.7	0.05
	B1/2	389	443	33.0	2.0	0.06
	B1/3	440	495	36.3	1.8	0.05
B2		323	372	8.9	0.9	0.10
B3		410	430	11.1	2.2	0.19
B4		334	384	9.7	0.6	0.06
B5	B5/1	423	457	19.7	0.8	0.04
	B5/2	422	444	19.4	0.9	0.05

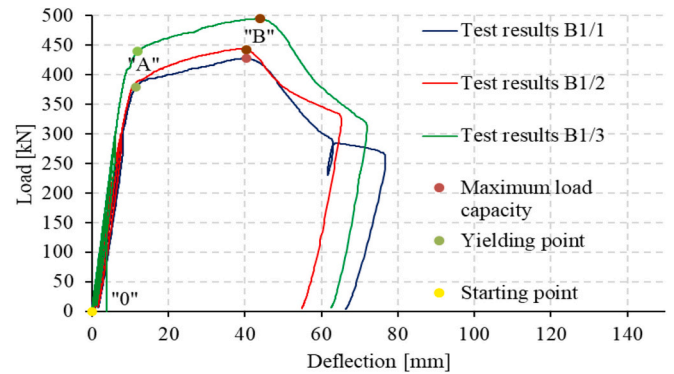


Fig. 8. Load-displacement diagram for B1/1 – B1/3.

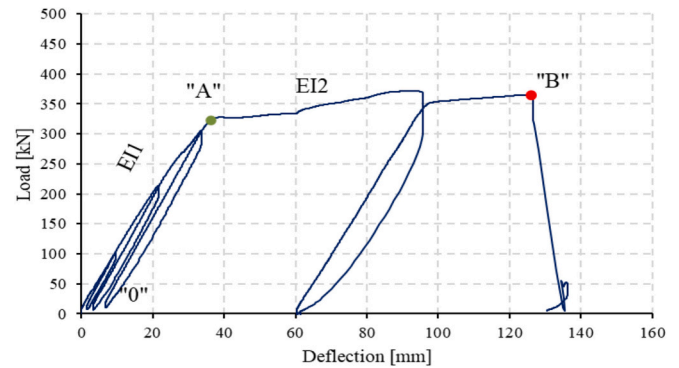


Fig. 9. Load-displacement diagram for B2.

- Initial stiffness is governed by the elastic state of the structural elements.
- At point "A" the lower steel flange yields and loses its stiffness for the further loads. Between points "A" to "B" due to this stiffness reduction the neutral axis of the cross-section shifted upwards

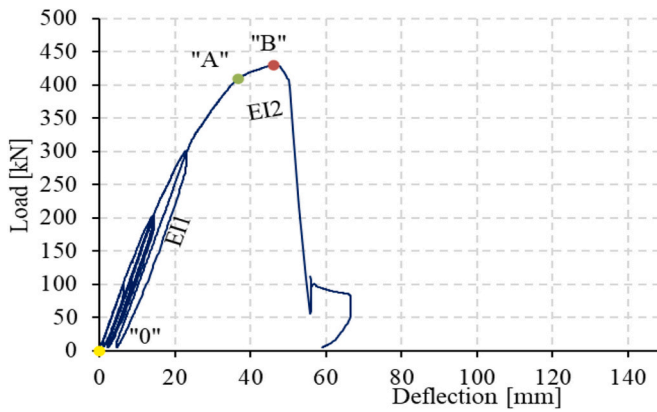


Fig. 10. Load-displacement diagram for B3.

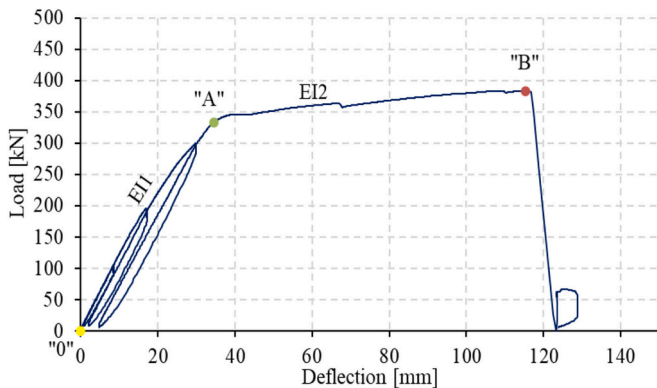


Fig. 11. Load-displacement diagram for B4.

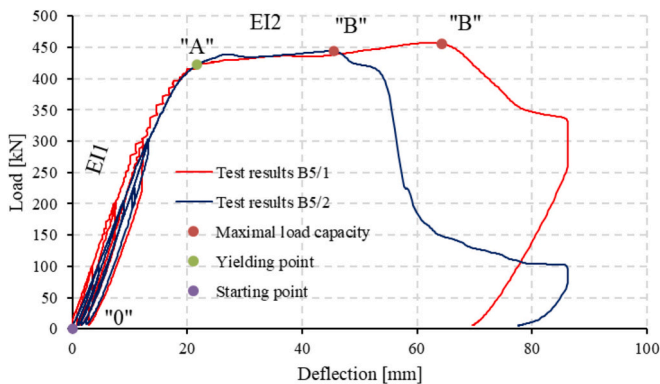


Fig. 12. Load-displacement diagram for B5/1 and B5/2.

resulting in the development of significant local bending moment in the upper concrete flange.

- (iii) At point "B" the concrete flange reaches its normal force and bending interaction load carrying capacity and suddenly fails by dominant normal force failure (column type failure). The failure is loud, since the concrete cover is suddenly removed by the buckled longitudinal rebars between the stirrups. Beside, the concrete core within the stirrups crushes significantly.
- (iv) No residual strength remains. The force suddenly drops to zero.

It is to be noted that the post-ultimate behavior (failure mode and residual strength) is significantly governed by the bending-normal force interaction in the upper concrete flange.

The **bending-shear interaction-type failure** is the mix of the above



Fig. 13. Shear failure of web in B1/1 (red dot presents inflection point). (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

described two failure modes, namely (i) moderate shear deformation develops without visible shear buckling in the web and (ii) the upper concrete flange is subjected to governing local bending moment with accompanying normal force thus not sudden but progressive column-type failure occurs (see later on in Fig. 28).

3.2. Shear type failure mode (specimens: B1 and B5/1)

Specimens B1/1-B1/3 and B5/1 show similar character in the force-displacement equilibrium path and in the collapse modes. The measured initial stiffnesses are between 20 and 36 kN/mm until reaching point "A". At point "A" the shear buckling of the web panel starts. Except B1/2 where local web crippling occurs at the support location. Between point "A" and "B" (hardening phase), due to progressive shear buckling of the web (shear deformation) the shear transfer between the upper concrete and lower steel flange is not or minorly provided thus the further load is mostly carried by the local bending of the flanges. Point "B" presents the maximum load value. At point "B" the upper concrete flanges reach their bending-normal force capacity and hinge develops in the concrete flange due to cracking and crushing.

In the case of B1/1 specimen, the load is introduced 1 m from the end support. Fig. 13 and Fig. 14 show the interactive shear buckling of the web and crushing of the upper concrete flange. In Fig. 13 a visible inflection point can be seen and marked by a red dot. Strain gauges are also attached to the top surface of the bottom steel flange. The results can be seen in Fig. 14. The maximum load is 428 kN at displacement of 41 mm. Fig. 15 presents the measured force – strain curve of the bottom steel flange of B1/1 at the load introduction section. The green dot represents the corresponding "A" point where nonlinearity starts.

In the case of B1/2 specimen, the span length is 7 m and the load is introduced 1 m from the end support. The damaged cantilever length is 1 m. Fig. 16 show the failure of the specimen. Due to insufficient vertical stiffener length local web crippling failure occurred at the support, since



Fig. 14. Upper flange crushing in B1/1.

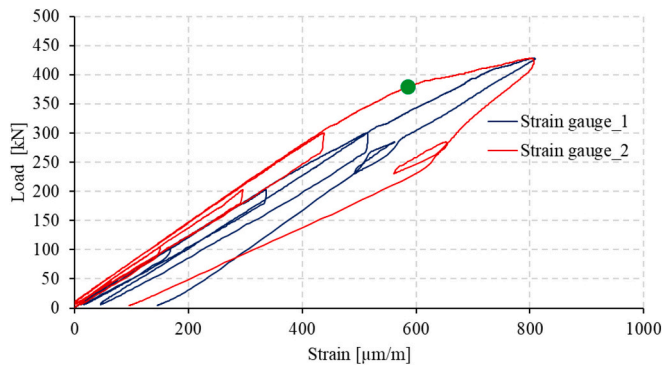


Fig. 15. Load-strain diagram for B1/1 specimen.

the stiffener was not attached to the flange by welding.

In the case of **B1/3** specimen, the span length is 6 m and the load is introduced 1 m from the end support. The damaged cantilever length is 2 m. Fig. 17 shows the failure of the specimen which shows the interactive shear buckling of the web and final crushing of the upper concrete flange due to dominant bending moment with accompanying minor normal force.

In the case of **B5/1** specimen, the load is introduced 1.33 m from the end support; span length equals 8 m. Failure mode shows the interactive shear buckling of the web and crushing of the upper concrete flange. The maximum load is 457,1 kN at displacement of 63 mm. These specimens failed by shear-type failure.

3.3. Bending type failure mode (specimens: B2, B3, B4)

Specimen **B2** is 8 m long, and it is loaded in the mid-span (4 m) as a three-point bending arrangement (Fig. 18). The girder behaves linear until reaching 322 kN where the lower steel flange yields and the neutral

axis is shifted up causing extra local bending moment in the upper concrete flanges for the further loads. At load level of 372 kN sudden column type failure of the upper concrete flange occurred. The buckling of the longitudinal rebars between the stirrups can be seen in Figs. 19 to 21.

Specimen **B3** is 8 m long and loaded 2 m apart from the support (Fig. 22 and Fig. 23). The girder behaves linear until reaching 410 kN where the lower steel flange yields and the neutral axis is shifted up causing extra local bending moment in the upper concrete flanges for the further loads. At load level of 430 kN sudden column type failure of the upper concrete flange occurs. The buckling of the longitudinal rebars between the stirrups can be seen in Fig. 22 and Fig. 23.

Specimen **B4** is also 8 m long and loaded 3 m apart from the support (Fig. 24 and Fig. 25). The girder behaves linear until reaching 334 kN where the lower steel flange yields and the neutral axis is shifted up causing extra local bending moment in the upper concrete flange. At load level of 384 kN sudden column type failure of the upper concrete flange occurred. The buckling of the longitudinal rebars between the stirrups can be seen in Fig. 26. Fig. 27 presents the load – strain diagrams of the bottom steel flange. The green dot represents the corresponding “A” point where yielding of the bottom steel flange starts.

3.4. Bending-shear interaction failure mode (specimens: B5/2)

In the case of the specimen **B5/2**, the span length is 6.67 m; cantilever length is 1.33 m. Load is applied 1.50 m apart from the support. The girder behaves linear until the load of 422 kN. The measured ultimate load is 444 kN. The failure of the upper concrete flange is similar to the bending-type failure mode but with progressive development of rebars buckling and concrete crushing (Fig. 28). On the other hand side no visible web shear buckling type failure occurred.



Fig. 16. Deformation of the specimen B1/2.



Fig. 17. Web buckling of B1/3; Concrete cracks above the support (close to the load).



Fig. 18. Applied load in mid-span (B2).



Fig. 19. Concrete flange failure (B2) – view from above.



Fig. 20. Concrete flange failure (B2) – view from below.

4. Evaluation of the results

All measured load-displacement equilibrium paths are comparatively presented in one graph, these are plotted in Fig. 29. The different failure modes are presented by different colors, namely bending-type failures (red), shear-type failures (green) and interaction failure (blue). In the case of bending type failure, the character of the hardening stiffnesses differ from the shear type failure; they show more flatter secondary stiffnesses. It is to be noted that in bending-type failure no residual strength remains after the maximum force value. The force suddenly drops to zero.

Fig. 30 presents the overall bending-shear interaction results. The



Fig. 21. Damage on the upper concrete flange (B2).



Fig. 22. Upper concrete flange failure (B3) – side view.



Fig. 23. Upper concrete flange failure (B3) – view from above.

horizontal axis represents the measured maximum bending moment values while the vertical axis represents the measured maximum shear forces (as given in Table 2). The red lines illustrate the hand calculated standardized resistances, namely the pure bending moment resistance of the flanges only and the pure shear buckling resistances of the corrugated web panels according to the Eurocode (see Eqs. (1)–(11)). Results prove that the upper concrete flange has notable contribution to the shear resistance. In addition, the points show significant bending-shear interaction which can be attributed to the bending-normal force interaction in the upper concrete flanges.



Fig. 24. Test setup of B4.



Fig. 25. Load introduction (B4).



Fig. 26. Concrete flange failure of B4 – view from above.

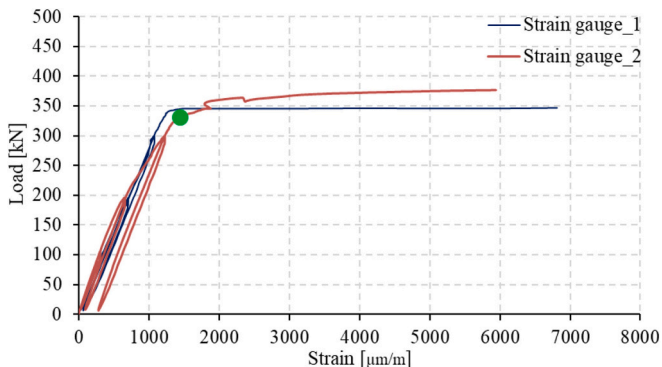


Fig. 27. Load-strain diagram of B4 specimen.



Fig. 28. Crushing of the upper concrete flange of B5/2.

Zijian Bi et al. published comparable experimental test results in [31]. They applied upper steel flanges below the upper concrete flange in order to connect the steel to concrete by embedded headed studs. Their experimental results are also plotted together with the current test results in Fig. 31.

The horizontal and vertical axes represent the bending and shear force ratios, respectively. For the reference resistance the Eurocode-based bending moment and web shear buckling resistances are applied according to Eqs. (1)–(11). Blue dots present the current results while red dots present the test results of Zijian Bi et al. [31]. It can be seen, that the two test results are aligned; they show similar character for the bending-shear interaction behavior. It can be also seen, that the test results show a significant flange contribution coming from the concrete slab, which could be utilized in the design equations. The lower bound curve of the test results are also shown on the figure, which could be a good bases for a design method development as a future research topic.

5. Conclusions

Current paper experimentally investigates the resistance of concrete-steel composite corrugated web girder. The main purpose of the research is the investigation of the integrated shear resistance of the composite cross-section under different bending moments and shear force interactions. In the frame of the research program 8 laboratory tests are performed and their results are evaluated. Three typical failure mechanisms are observed, namely (i) bending-type failure, (ii) shear-type failure, (iii) bending-shear interaction failure. Based on the test results the following conclusions are drawn out:

- The upper concrete flange contributes significantly (14–44 %) to the shear resistance of the composite cross-section compared to a pure steel girder.
- The upper concrete flange is subjected to the interaction of normal force and bending moment. The collapse mode of the concrete flange depends on the normal force and bending moment ratios, as proved by the test results.
- Due to shear buckling of the web, the shear transfer is not ensured between the flanges and the upper concrete flange (slab) gets dominantly subjected to local bending. Progressive failure is identified.
- In the case of low shear utilization ratio (corrugated web is in elastic state), the upper concrete flange is dominantly subjected to normal force. Sudden failure is identified.
- Preliminary lower bound design proposal is shown based on the experimental test results extended by test results taken from the international literature.

The results are limited to the parameter domain of the experimental tests therefore the future aim of the authors is to develop and validate an

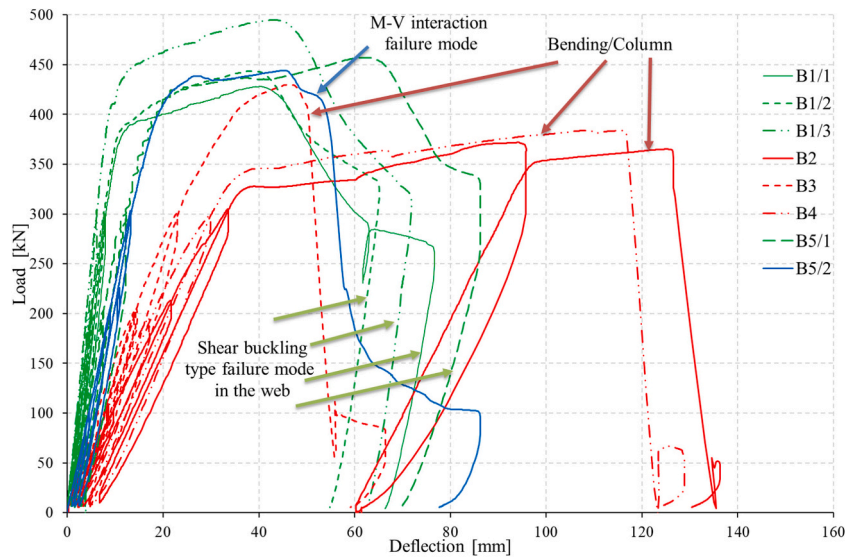


Fig. 29. All specimen's force-displacement curve for comparison.

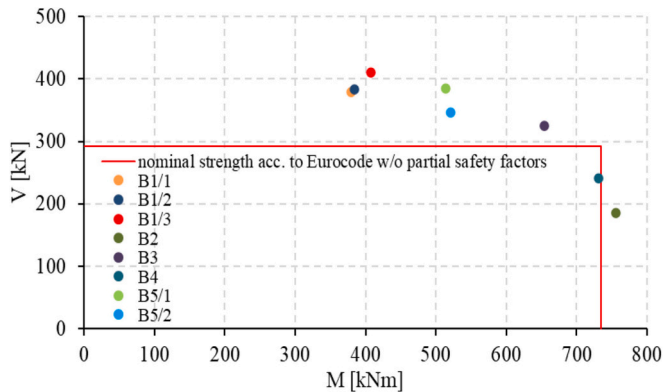


Fig. 30. M-V interaction.

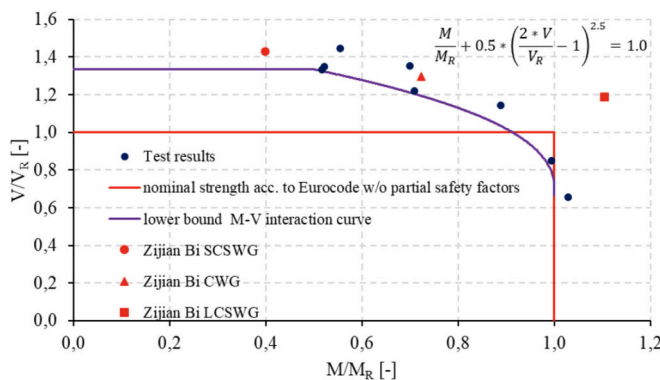


Fig. 31. Results compared to results from literature [28] results.

advanced numerical model in order to extend the investigated parameter domain. Based on the numerical results an improved design proposal will be developed and presented.

CRediT authorship contribution statement

Bence Jäger: Supervision, Methodology, Investigation, Conceptualization. **Tibor Pap:** Writing – original draft. **Balázs Kövesdi:** Writing –

review & editing, Supervision, Methodology, Funding acquisition, Conceptualization. **László Dunai:** Writing – review & editing, Supervision, Methodology, Funding acquisition, Conceptualization.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

Data will be made available on request.

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