Resistance of Steel – Concrete Composite Girders with Corrugated Web

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Abstract. The paper discusses the analysis of shear resistance of girders with corrugated web, combined with a concrete slab in a way that allows creation of a composite structure according to the definition of Eurocode 4. The results of the authors’ experimental tests of girders with a corrugated web connected to a concrete slab, and the results of tests on girders without a concrete slab were used. Experimental investigations concerned the effectiveness of SIN girder (i.e. with sinusoidal corrugation) WTA 500 / 300x15 strengthening with a concrete slab for shear resistance of the whole steel-concrete element and shear buckling resistance of the web. Load – displacements paths LDPs \( P(y) \) as well as the results of tensometric strain measurements of the web were analysed. The results of the composite girder tests were compared with the results of tests of SIN girder without a concrete slab with a similar static diagram and a similar failure mode due to the loss of the web stability. In addition, the first buckling load and slab contribution to shear force transferring were analysed. Based on the analysis of experimental results and available analytical models, the solution for estimating design shear buckling resistance in girders with corrugated web strengthened with a concrete slab was proposed. Using the proposed formula, the contribution of the web and concrete slab to the transfer of shear load for the whole series of girders with web heights \( h_w = 500 \text{ mm} \) to \( 1500 \text{ mm} \) and thickness \( t_w = 2, 2.5 \text{ and } 3 \text{ mm} \) was shown. The proposed formula takes into account the mutual interaction between the local and global mode of shear stability of the web, which leads to the reduction in design shear buckling resistance. Additionally, improvement in the web boundary conditions as an effect of increased flange stiffness due to joining it to the concrete slab was taken into account. This paper also uses the results of tests concerning SIN girders without a concrete slab, with other web heights and thicknesses than in WTA 500/300x15.

1. Introduction
SIN type plate girders with sinusoidal corrugated web became popular at the end of the 20th century. Currently, thin-walled corrugated webs with thicknesses \( t_w =2, 2.5 \text{ and } 3 \text{ mm} \) (although it is also possible to fabricate webs that are thicker than the ones above), heights ranging from 333 to 1500 mm and wave amplitude of 40 mm are utilised. Compared with flat webs in traditional plate girders, in thin-walled corrugated webs shear stress is utilised to a greater extent. As a result, lighter structures can be built. In SIN girders, however, like in other plate girders, the phenomenon of shear instability of the web is found. The phenomenon often produces girder resistance reduction below the theoretical resistance value that results from pure bending [1-4]. In Eurocode 3, local and global shear instabilities of the web panel are considered separately. By contrast, for some time, the literature on the subject has reported theoretical
models that make it possible to account for interactive character of the web stability loss when specifying its design resistance [5-8].

In steel-concrete composite girder, a part of the shear force is transferred by the slab. In addition, the conditions of support and the web wall loading are changed compared with the steel girder web [9]. Eurocode 4 [10] recommends that shear resistance of steel-concrete composite elements should be determined based on Eurocode 3 [11]. The rules given in the code [11] are intended for structural solutions in which the web panel can be treated as simply supported on the edges. The support conditions of a web mentioned above are found in steel girders with flanges and stiffeners with low torsional stiffness, and it differs from the support conditions in composite girders.

Although [10] allows taking into account slab interaction in shear, the guidelines on the computational model are not provided. Study [9] reported that the application of concrete slab connected with SIN girders substantially increased their shear resistance. In this study, an initial, greatly simplified method for resistance estimation was employed. The method involves a simple summation of resistances of the web of not strengthened steel girders obtained from the tests and the slab shear resistance acc. the Polish standard for concrete structure design. This study makes an attempt at analysing the issue in a more accurate way, in which different beam heights are taken into account. A modified method for the estimation of design buckling resistance [8] was used. The method, different from the one given in Appendix to the standard [11], is based on the determination of interactive buckling resistance.

2. Experimental investigations
The tests were conducted on girder WTA500/300x15 with corrugated web, 2 mm in thickness (made from nominal steel grade S235JRG2), which was bonded to a concrete slab, 12 cm in thickness and 30 cm in width (figure 1). The connection between the steel and concrete parts was made using bars ø16. They were of inverted U-shapes, welded to the top girder flange 10 cm apart. In the concrete slab, longitudinal or transverse reinforcement was not used. Concrete modulus was specified as $E_{cm} = 33$ GPa, compressive strength as $f_{cm,cyl} = 45.6$ MPa and tensile strength as $f_{ctm} = 2.78$ MPa.

![Figure 1](image)

Figure 1. Composite girder B-1 WTA500 with a concrete slab – dimensions and static diagram

The girder (figure 2) was loaded with force $P$ by means of the hydraulic press (1) attached to the frame (FR). A dynamometer was placed between the piston and the girder of concern. Web strain $\varepsilon$ was measured using an array of strain gauges (3), and the beam vertical displacements $y$ were controlled by means of induction sensors located in the span (4) and on the supports (5) and (6). Strain was also measured along the flange and the concrete slab. Support reactions at points (5) and (6) were transferred by the support stiffeners onto the bearings. The rate of load increase until the occurrence of the onset of clear yield zone formation in the web was approx.18 kN/min.
The maximum value of force was $P_u = 459$ kN, and the failure process was initiated by the web instability, as it was the case with steel girders [2], [12]. As the yield lines, visible to the naked eye, started forming in the web, a crack developed in the concrete slab. The crack was located at the extension of the steel stiffener, above the hinge movable support, in the direction transverse to the beam axis. At the end of the tests, the connection failed because of concrete, which manifested itself as the slab cracking along the line of shear connectors (figure 3a).

It should be emphasised that the concrete slab failure occurred together with the yield zone formation, i.e. when the corrugated web resistance reserve was utilised. Longitudinal slab cracking results from a rapid increase in the debonding force acting on the connection with the slab (figure 3b). The mechanism of the steel - concrete connection behaviour at the instant of failure resembled that occurring in composite beams with openings. A transverse reinforcement (possible at greater slab width) and longitudinal reinforcement would allow an increase in the beam ductility. Due to small span of the girder, the limit load $P_u$ produced a bending moment that was equal to about 50% of limit bending resistance.

Tests on steel girders without a reinforced concrete slab, discussed in studies [2] and [12], which were employed for the sake of comparison, were conducted in a manner similar to the tests described in this section. Dimensions and loading diagrams of those girders are shown in Table 1.

Figure 2. Girder B-1: a) on the test stand, b) strain gauge rosette arrangement on the web

(a) (b)

Figure 3. Girder B-1: a) failure mode, b) slab behaviour after instability; 1 – concrete strut

(a) (b)
3. Strains in the girder web
To specify the onset of instability in the corrugated web, the profile of strains was analysed using strain
gauges diagonally glued onto the surface of the web at points R1÷R8 (see figure 2b). Points of the
corrugated web stability loss and the corresponding buckling load $P_{cr}$ in steel-concrete composite girder
were marked in figure 4a. To compare strains, a similar graph for one of steel girders without the slab
was provided (figure 4b).

In the composite girder, lower mean values of the web strain relative to the web thickness, under
the same shear force, were measured than it was the case for steel girders. In the linear range (up to $P_{cr}$), for
girders with the web height of 500 mm, this difference was approx. 10%, which indicates that portion
of the force was taken by the slab (see also Section 5). Strain values measured in the main direction (~45
degrees) under load $P_{cr}$ in the composite girder range is $\varepsilon \approx 0.64 – 0.84\%$, whereas in non-composite
girders $\varepsilon \approx 0.50 – 0.67\%$, i.e. up to 28% less. In all girders, the point of the web stability loss was found
at strains below $1\%$.

Figure 4. Strains in the main direction relative to the centre axis: a) girder B-1, b) girder BS-3

In steel beams, the onset of a rapid increase in measured strains at points R1÷R7 (see figure 4b) was
associated with the start of the formation of the clearly visible yield zone [8, 12]. In the composite beam,
the yield zone developed much slower, however, that did not mean linear-elastic range of the web
behaviour. Since in the composite beam the web deformation that can be seen with a naked eye occurred
only at the end of the tests, buckling load was estimated on the basis of the analysis of the dependence
$V/V_0 = \varepsilon/V$ (figure 5). First signs of the web nonlinear behaviour were recorded at the load level of $P = P_{cr} = 340$ kN (rosette at point R5, and a moment later at point R1 and R8). The complete formation of the
tension field, though it was still not visible with a naked eye, probably occurred for the load level of
$P = N_{Q_{rys}} = 383$ kN, as earlier reported in study [9].

Figure 5. The method of determining the first buckling load $P_{cr}$ in the BS-1 girder
4. Load – displacements paths (LDPs)

Based on the measurements of the global displacement $y$ and load $P$ obtained from experimental investigations, load displacement paths LDPs $P(y)$ in composite (strengthened with slab) girders and not strengthened ones, i.e. without a slab, were determined. In figure 6, selected LDPs $P(y)$ of girders: B-1 with a composite flange and BS-1 without a slab are shown. In the graph, clear nonlinearity of LDPs $P(y)$ occurred after the exceedance of the first buckling load $P_{cr}$, although the end of ideally linear $P-y$ relation in the composite girder was found slightly earlier (see figure 6a).

![Figure 6](image)

**Figure 6.** Comparison of LDPs $P(y)$ from experimental investigations: a) girder B-1, b) girder BS-1

| Girder $(h_w \times t_w)$ | Static diagram of girder | Limit Load $P_{ur}$ | First buckling load $P_{cr}$ | $P_{cr}$ | $V_{cr}$ | Comments |
|--------------------------|--------------------------|----------------------|-----------------------------|---------|---------|----------|
| B-1 (500x2)              | ![Diagram](image)        | 459                  | 340                         | 0.74    | 170     | authors’ research |
| BS-1 (500x2)             | ![Diagram](image)        | 347                  | 210                         | 0.61    | 105     | research acc. [2] |
| BS-2 (500x2)             | ![Diagram](image)        | 357                  | 215                         | 0.60    | 108     | research acc. [2] |
| BS-3 (500x2.5)           | ![Diagram](image)        | 436                  | 280                         | 0.64    | 140     | research acc. [12] |

* The force corresponding to the beginning of the web stability loss

Table 1. Experimental results for composite and steel girders.
In load-displacement paths, the coordinates of the characteristic points, namely \( P_1(P_{cr}) \) and \( P_2(P_{uR}) \) were marked. The notations refer to:

- \( P_1(P_{cr}) \) – the web stability loss signalled by the occurrence of diagonal yield zones, which leads to crack formation in the concrete slab of the composite girder. The point corresponds to the first buckling load \( P_{cr} \);
- \( P_2(P_{uR}) \) – the limit load from the condition of the global girder failure \( P_{uR} \) signalled by the end of the tension field formation and failure of the concrete slab in the composite girder.

In the global LDP \( P(y) \), the linear range of displacements is marked as \( 0 - P_1(P_{cr}) \). In the range \( P_1(P_{cr}) - P_2(P_{uR}) \), a strong effect of elastic-plastic displacements produced by shear forces in the web on the girder total displacements is observed. The effect is associated with crack formation in the concrete slab.

In the composite girder, the range of girder elastic strains \( 0 - P_1(P_{cr}) \) increased. The occurrence of a substantial shift of the coordinate \( P_1(P_{cr}) \) indicated an increase in the buckling load \( P_{cr} \).

Experimental results for girders that failed because of shear stability loss are compiled in Table 1. Column 3 shows limit resistance \( P_{uR} \) measured by force \( P \), whereas column 4 lists elastic buckling resistance \( P_{cr} \) measured by force \( P \). For comparison, experimental results for corrugated web beams without a slab were also provided [2], [12]. Buckling load in the composite girder B-1 was greater compared with not strengthened girders BS-1 and BS-2 by from 58% to 62%. For girder BS-3, after the difference in web thickness was taken into account, an increase by 52% was found. The limit load increased from 29% to 33%.

5. Shear load separation into the corrugated web and the concrete slab

The total shear force \( V_{cr} \) transferred jointly by the web \( V_w \), top flange bonded to the concrete slab \( V_{fT} \) and bottom flange \( V_{fB} \) constitutes a half of the buckling load \( P_{cr} \) (see Table 1). The estimates of the shares of individual section parts in transferring shear force at the instant of the onset of the web stability loss at point \( P_1(P_{cr}) \) were given based on modified theoretical dependences acc. [13]. For girders with the same slab thickness as the examined girder B-1, shear force transmitted by the web \( V_w \) and by the top flange bonded to the reinforced concrete slab \( V_{fT} \) was estimated from formulas that hold for the range of geometric linearity, namely (1) and (2):

\[
V_w = \frac{P_{cr} \cdot a}{L} \left( \frac{I_c - I_{ac} - I_{pd}}{I_c} \right) \frac{h_o}{h_o} \tag{1}
\]

\[
V_{fT} = \frac{P_{cr} \cdot a}{L} \frac{I_{ac}}{I_c} \left( 1 + \frac{I_c - I_{ac} - I_{pd}}{I_{ac}} \cdot \frac{y_o}{h_o} \right) \tag{2}
\]

where: \( P_{cr} \) – first critical buckling load, \( I_{ac} \) – moment of inertia of the composite flange (concrete slab + flange of the steel girder), \( I_{pd} \) – moment of inertia of the bottom flange, \( I_c \) – moment of inertia of the whole composite section (steel girder + concrete slab), \( a \) – load \( P \) distance from the support, \( L \) – span of girder, \( h_o \) – web height, \( h_o \) – distance between the bottom flange axis and the centre of weight of the composite flange, \( y_o \) – distance between the centre of weight of the composite flange and the web upper edge.

To estimate separated shear forces \( V_w \) and \( V_{fT} \) in girder B-1, it was assumed that concrete elastic modulus \( E_{cm} \) is 33 GPa and steel elastic modulus is 205 GPa. At the instant when buckling load \( P_{cr} \) was reached in girder B-1 strengthened with a concrete slab, shear force in the web, calculated acc. formula (1) was \( V_w = 151.9 \ \text{kN} \), whereas the force transmitted by the composite flange was \( V_{fT} = 15.8 \ \text{kN} \). That indicates shear force increase in the web at the instant of buckling load occurrence in girder B-1 strengthened with a concrete slab ranged from 41% to 45% compared with not strengthened girders BS-1 and BS-2. For girder BS-3, when the difference in web thickness is taken into account, the increase was 36%.
6. The proposal for determination of shear buckling resistance of the web of girders strengthened with a concrete slab

Profiles of strains recorded by diagonal strain gauges, the characteristics of load-displacement paths and of modes failure in composite girders and not strengthened steel girders indicate that in both cases the web failure mechanisms are similar. That means in girders strengthened with a concrete slab, the web stability loss mode, typical of corrugated web girders is still found. By contrast, total buckling loads $P_{cr}$ and shear forces transmitted by the web are substantially higher. As a result, to estimate design shear buckling resistance of the web, a modified computational model based on the calculations of interactive shear buckling resistance acc. [8] was employed. The modifications to the model involved taking into account greater buckling load transferred by the web of a composite girder. It was assumed that a part of concrete slab interacts in the transfer of shear force. The slab width involved is not greater than the width of the flange of steel girder (increase in the slab width does not cause an increment of buckling force in the web). Interactive shear resistance was determined on the basis of formula (3) acc. [8]:

$$\tau_{cr,6} = \frac{\tau_{cr,L} \cdot \tau_{cr,G}}{\left(\tau_{cr,L}^6 + \tau_{cr,G}^6\right)^{1/6}}$$  \hspace{1cm} (3)

where: $\tau_{cr,L}$, $\tau_{cr,G}$ – local and global shear buckling stress determined on the basis of the classical equations of the stability theory.

Using the determined interactive shear buckling resistance (3), interactive slenderness was estimated from formula (4):

$$\lambda_{I,6} = \sqrt{\frac{\tau_y}{\tau_{cr,6}}}$$  \hspace{1cm} (4)

where $\tau_y$ – shear yield strength.

Ultimately, design shear buckling resistance was estimated based on shear stress and interactive slenderness acc. formula (5):

$$\tau_{n,RT} = \tau_y \left[ \frac{2}{\lambda_{I,6}^6 + n} \right]^{1/6}$$  \hspace{1cm} (5)

In formula (5), an experimental coefficient $n = 2$ was adopted. By contrast, for steel girders, not bonded to the slab, the coefficient was $n = 5$ as in study [8]. A different value of the coefficient is related to a change in the web boundary conditions (flange stiffening with a concrete slab).

7. Results and discussions

In steel-concrete composite girders with corrugated web, stability loss of the corrugated web that ends with yield zone formation leads to the occurrence of cracks in the concrete slab. Shear buckling resistance of the corrugated web in the girder strengthened with a concrete slab, obtained through the tests, was estimated from formula (6). The results are listed in Table 2:

$$\tau_{INV} = \frac{V_w}{h_w t_w}$$  \hspace{1cm} (6)

where: $h_w$, $t_w$ – web height and thickness.

In order to show the concrete slab impact on the values of shear forces transferred by the web and the slab, the values of shear buckling resistance were analysed based on the proposed computational model. That was done for the whole range of fabricated girders with the height $h_w = 500 – 1500$ mm and web thickness $t_w = 2$, 2.5 and 3 mm. In the analysis, the girder yield strength was assumed $f_y = 266.2$ MPa in accordance with the material tests on girder BS-3 acc. [12].
Firstly, shear buckling resistance of the webs $\tau_{n,BA}$ was determined for the whole series of girders. Based on shear buckling resistance $\tau_{n,BA}$, shear forces $V_w$ transferred by the web in composite girders was estimated from Eq. (6). Next, the magnitude of shear forces transferred by the concrete slab $V_{fT}$ and force $V_w$ transmitted by the web were estimated acc. Eqs. (1) and (2).

Table 2 summarises the results of computations. It also shows a comparison of shear buckling resistance and shear forces transferred by the corrugated web and the concrete slab. The comparison was based on experimental investigations and, for the whole series of SIN girders (with flange dimensions 300 mm x 15 mm), on the solution proposed by the authors $\tau_{n,BA}$ (5).

Table 2. Predictions of design shear buckling resistances of girders with concrete slabs.

| Type  | $\tau_{INV}$ test [MPa] | $\tau_{n,BA}$ (5) [MPa] | $V_w$ test [kN] | $V_{fT}$ test [kN] | $V_w$ calc. [kN] | $V_{fT}$ calc. [kN] |
|-------|-------------------------|--------------------------|-----------------|-------------------|-----------------|-------------------|
| 500x2 | 151.9                   | 153.5                    | 15.8            | 153.5             | 15.8            |
| 500x2.5| 153.6                   | 192.1                    | 20.0            |
| 500x3 | 153.7                   | 230.5                    | 24.0            |
| 625x2 | 153.5                   | 191.9                    | 15.5            |
| 625x2.5| 153.6                   | 268.1                    | 23.3            |
| 625x3 | 153.7                   | 240.1                    | 19.4            |
| 750x2 | 153.5                   | 288.0                    | 19.1            |
| 750x2.5| 153.6                   | 345.7                    | 22.9            |
| 750x3 | 153.6                   | 306.1                    | 14.8            |
| 1000x2| 153.1                   | 383.2                    | 18.6            |
| 1000x2.5| 153.4                   | 460.1                    | 22.3            |
| 1000x3| 151.5                   | 378.8                    | 14.5            |
| 1250x2| 152.1                   | 475.4                    | 18.1            |
| 1250x2.5| 152.5                   | 571.8                    | 21.8            |
| 1500x2| 147.8                   | 443.4                    | 14.0            |
| 1500x2.5| 149.3                   | 560.0                    | 17.6            |
| 1500x3| 150.3                   | 676.3                    | 21.3            |

Figures 7 and 8 illustrate a change in the magnitude of the shear forces transferred by web $V_w$ and the slab together with the top flange $V_{fT}$ in a series of corrugated web girders strengthened with a concrete slabs (dimensions of slabs adopted in calculation were the same as those of the examined girder B-1). The mentioned change was expressed as a function of the web height. The share of the concrete slab in transferring shear load $P_{cr}$ is up to 10% in low girders at $h_w = 500$ mm and $t_w = 3$ mm. This share drops, in a non-linear manner, with the web height to 3% in high girders at $h_w = 1500$ mm and $t_w = 2$ mm. Therefore, concrete slab contributes to the transfer of the total shear load $V$ pertaining to the composite girder. Additionally, the slab improves stability of the compression flange, which substantially enhances both shear buckling resistance and limit resistance.

The values of shear stress obtained for the composite girder B-1 in accordance with formula (1) and (6) were 151.9 MPa and 153.5 MPa. For comparison, mean stress estimated on the basis of averaged readings from eight strain gauges, glued diagonally on the surface of the web (0.738 ‰), under force $P_{cr} = 340$ kN was almost the same – 151.3 MPa (from 130.8 to 173.2 MPa), and after taking into account the wave unfolding – 147÷195 MPa.
In addition to specifying buckling resistance of the web, it is also necessary to state the resistances of the reinforced concrete slab and shear connectors, because premature failure of the slab or the connection makes it impossible to obtain the predicted increase in the web resistance. Therefore, condition (7) should be checked:

\[ V_{ft} \leq V_{R,slab} \]  

(7)

The means of specifying the slab resistance \( V_{R,slab} \) for both the absence and presence of cracks, and also the stress level of shear connectors in the beams in which the web instability did not occur were reported in study [14]. When determining the resistance of the whole system, the behaviour of shear connectors was treated as that of vertical reinforcement. Because shear post-buckling resistance is not taken into account in design, the means of making computations adopted in [14] can also be employed for slabs in girders with corrugated web.
8. Conclusions
Corrugated web girders strengthened due to their connection to a concrete slab show relevantly higher shear resistance than their not strengthened counterparts. However, the effectiveness of strengthening slightly decreases with an increase in the girder height or the web thickness. Resistance enhancement is produced, to a lesser extent, by taking a portion of the shear force by the slab and, to a greater extent, by improvement in the boundary conditions of the web. The improvement results from a substantial increase in torsional stiffness of the flange resulting in the stiffening of the web edge. The application of a concrete slab causes not only increase in the global buckling force, but also extends the linear-elastic range of the load – deflection relation. Additionally, that slows the increment of plastic deformation.

The analyses presented in the paper are intended to probe into the subject matter. The means of computations proposed in the study cannot be considered as a final solution to the problem. To develop final computational procedures, it is necessary to carry out further investigations, the range of which should be similar to those conducted for not strengthened girders, e.g. [2, 12]. The authors think the results obtained could be mainly used to evaluate the effectiveness of the potential structural strengthening. Concrete slab bonded to a steel beam can be regarded as an alternative method for enhancing shear resistance in composite beams with corrugated web, which was reported, e.g. in study [8].

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