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A new design proposal for composite beams with powder actuated shear connectors

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Abstract. This paper introduces a proposal for the SP 266.1325800 code update, which is applied as a national building code for composite structure design in Russian and some CIS countries. The proposal is based on deep codes analyze and comparison and previous authors' publications. The research is dedicated to nailed shear connectors' behavior in composite beams with profiled sheeting. The proposal provides a less conservative approach for composite beams with ductile shear connectors and takes into account additional conditions, such as decking geometry and the structure's performance under fire. The proposed methodology defines the main geometric characteristics of the composite cross-section before determining the shear connector type. If ductile connectors are chosen, it is recommended to use partial design methods for shear connection definition due to the economic benefits of the proposal. The proposal is verified on powder-actuated shear connectors and should be double-checked with other working examples and connector types. The results show that the structure designed according to the new proposal is 13% less utilized with a smaller quantity of shear connectors (22% less) under the same load. It provides benefits from the structure costs and labor time.

1 Introduction

Composite floors, made of steel beams connected with concrete slabs by shear connectors, are wildly used in construction practice for decades. In terms of shear connection design, there are two proposals exist: composite beams could be considered as fully or partially connected.

There is total shear resistance in structures with full connection overwhelms the longitudinal shear force, which takes place due to bending of composite structure parts. Slip and slip strain are everywhere zero, and it can be assumed that cross-sections remain plane [1]. This approach is usually used for elastic analyze and provides most conservative results. It could be useful for design of structures applying intensive dynamic loads (e.g. bridges) or preliminary calculations [2].

Another approach, that uses partial shear connection in beams, is based on two independent studies from early 90s [3] and preferable for civil structures. Such floors include connectors which provide less shear resistance that force is applied. It evokes some end-slip restrained by deformation of shear connectors. It makes designer use plastic

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analyze for calculation. For the safety requirements matching, there are two groups of conditions should be met.

The first group is related with ductile longitudinal shear behaviour. There are some options for ductility definition, however the most common is EN 1994-1 [4] approach. The shear connector should be exanimated by standard push-test that provides data for characteristic slip of the connectors. If it exceeds 6 mm, the connectors could be considered as ductile and use for structures with partial connection [5].

The second group of conditions is related with the degree of shear cconnection, which is defined by the ratio $\eta=N_c/N_{c,f}$ (where N_c is design value of the compressive normal force in the concrete flange; $N_{c,f}$ is compressive normal force in the concrete flange with full shear connection. The η should more or equal than 0.4 exclude floors under the seismic load. There is $\eta \geq 0.8$ for this case [6]. Another requirement, that should me matched is:

$$\eta > 1 - \frac{355}{R_y}(0.75 - 0.09L) \tag{1}$$

where
R_y – Nominal value of the yield strength of structural steel;
L – Span length.

There is SP 266.1325800 [7] is applied as a national building code for composite structure design in Russian and some CIS countries. Despite of the fact, that the code is based on the same design principles as EN 1994-1, it doesn`t include any provisions for partial connections design [8]. As the result, structures, which are designed according to the standard, aren`t so efficient as could be.

The paper follows the research which is dedicated to nailed shear connectors behaviour in composite beams with profiled sheeting and introduces the proposal for the SP 266.1325800 code update. The proposal is based on deep codes analyze and comparison and previous authors` publications [6, 9, 10].

2 Methods

During the previous stages of the research there some gaps in the SP 266.1325800 code were defined:
- no criterion for ductile/non-ductile shear connectors.
- partial connection for composite beams is not covered.
- reduction factor is not optimal for X-HVB-type connectors.
- shear resistance reduction in terms of fire for any type of connectors is not considered.

The initial stage of the investigation included 15 series of push-tests with Hilti X-HVB with 11 of them which were taken for estimation and analysis. The test results have been also compared with data from similar research made by other authors [11-13]. It was confirmed that X-HVB connectors could be considered as ductile according the EN 1994-1 criterion. Due to the fact, that SP 266.1325800 and EN 1994-1 are based on the same principles, the criterion was applied for the proposal.

The design proposal is based on SP 266.1325800, the whole algorithm is shown on figure 1: steps which are marked with blue line were supplemented with some new provisions based on authors early investigations; steps which are marked with red line were added for the first time and based on EN 1994-1 proposals.

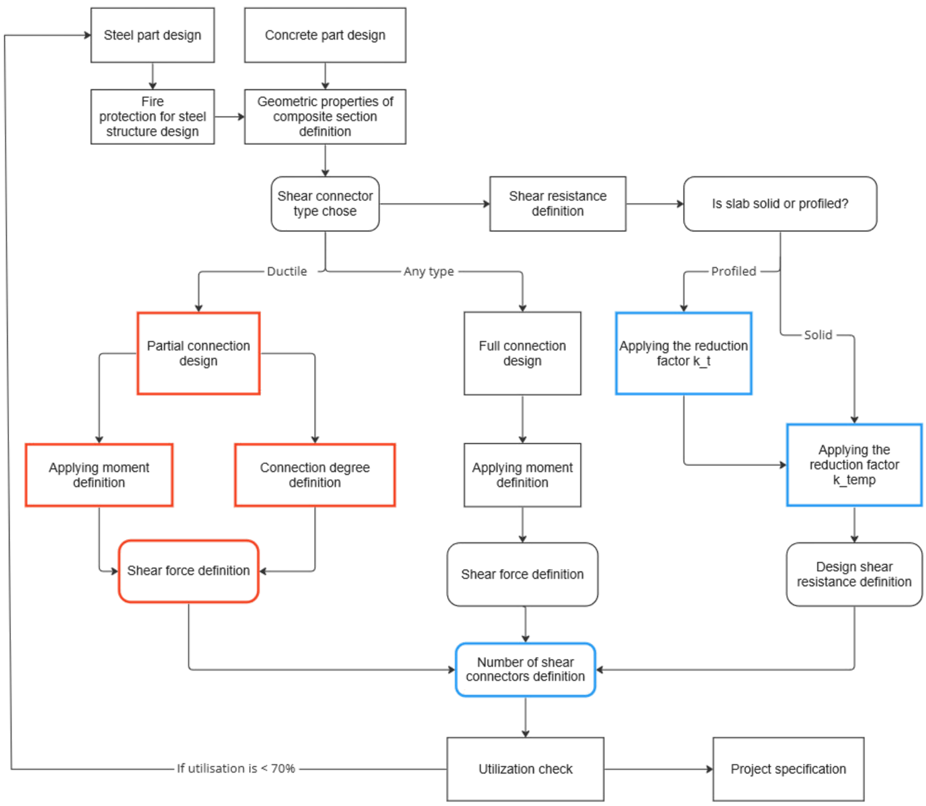

Fig. 1. Algorithm for the design proposal

According to the proposed methodology, before shear connection definition, the designer determines the main geometric characteristics of the composite cross-section. After that, it is necessary to determine the shear connector type: if ductile connectors are chosen, it is recommended to use partial design method for shear connection definition due to economic benefits of the propose.

If there is partial design method with plastic analysis is chosen, the applying bending moment for critical cross-section depends on plastic neutral axis (PNA) allocation. When it is in concrete slab ($N_b \geq N_{st}$):

$$M_{pl,Rd} = R_y \cdot A_{st}\left(\frac{h_{st}}{2}\right) + R_b b_{sl}\left(t_{sl} - \frac{h_c}{2}\right) \tag{2}$$

where
A_{st} – Cross-sectional area of the structural steel section;
h_{st} – Depth of the structural steel section;
R_b – Design value of the cubic concrete compressive strength;
b_{sl} – Effective width of steel flange;
t_{sl} – Thickness of a flange of the structural steel section;
h_c – the compressed zone height in the concrete, determined by the formula:

$$h_c = \frac{R_y \cdot A_{st}}{R_b b_{sl}} \tag{3}$$

When PNA is in flange of steel beam ($N_b \leq N_{st}$ and $N_b < R_y \cdot s_{st} \cdot (h_{st} - 2t_{st})$):

$$M_{pl,Rd} = R_b \cdot b_{sl} \frac{(h + t_{sl})^2}{2} + R_y \cdot A_{st} \cdot \frac{h_{st}}{2} - 2R_y \cdot b_{st} \cdot \frac{{t_{st}}^2}{2} - \\ -2R_y \cdot s_{st} \left(\frac{x^2 - {t_{sl}}^2}{2} \right) \tag{4}$$

where
h – Thickness of a concrete slab above the deck;
s_{st} – Thickness of a web of the structural steel section;
x – The compressed zone height in the steel beam, determined by the formula

$$x = \frac{R_y \cdot A_{st} - R_b \cdot b_{sl}(h + t_{sl}) - 2R_y \cdot t_{st}(b_{st} - s_{st})}{2R_y \cdot b_{st}} \tag{5}$$

When PNA is in flange of within web of steel beam ($N_b \leq N_{st}$ and $N_b < R_y \cdot s_{st} \cdot (h_{st} - 2t_{st})$)

$$M_{pl,Rd} = R_b \cdot b_{sl} \frac{(h + t_{sl})^2}{2} + R_y \cdot A_{st} \cdot \frac{h_{st}}{2} - 2R_y \cdot b_{st} \cdot \frac{x^2}{2} \tag{6}$$

The difference between compressive forces in the slab and tension forces in the steel beam must be balanced by the linear shear forces from tangential stresses along the contact surface in order to keep the balance of internal forces in composite structure:

$$\sum S_i = \min(R_b \cdot A_b, R_y \cdot A_{st}) \tag{7}$$

where
A_b – Cross-sectional area of concrete.

When the minimum connection degree is defined, the relevant compressive force in concrete N_b could be calculated with:

$$N_b = \eta \cdot N_{b,f} = \eta \cdot R_b \cdot b_{sl} \cdot t_{sl} \tag{8}$$

where
$N_{b,f}$ – Design value of the con1pressive nominal force in the concrete flange with full shear connection.

If the longitudinal shear force between steel and concrete within points of zero and maximum moment does not exceed N_{st} or N_b, then the composite beam is considered fully connected ($\eta = 1$). Consequently, the following equality is valid for the structure with partial connection:

$$\eta = \frac{\sum S_i}{N_b} = \frac{\sum P_{rd}}{N_{st}} \tag{9}$$

where
$\sum P_{rd}$ – Design value of the shear resistance of all connectors;
N_{st} – Design value of norn1al force in steel section.

The following step is design shear resistance of connectors definition and calculating the total amount of connectors. Total design shear resistance of connectors is:

$$\sum P_{rd} = n \cdot P_{rd} \cdot k_t \tag{10}$$

Substituting the obtained value in (9), we determine the actual connection degree for the structure with calculated connectors amount:

$$\eta = \frac{n \cdot P_{rd} \cdot k_t}{R_y \cdot A_s} \tag{11}$$

The bearing capacity of the composite beam in the plastic stage with partial connection could be determined by the formula:

$$M_{Rd} = R_y \cdot W_{pl,x} + \eta\left(M_{pl,Rd} - M_{pl,\text{a},Rd}\right) < M_{pl,Rd} \tag{12}$$

where
$M_{pl,\text{a},Rd}$ – Design value of the plastic resistance moment of the structural steel section;
$M_{pl,Rd}$ – Design value of the plastic resistance moment of the composite section with full shear connection.

The deflection of a steel-reinforced concrete beam floor, provided it is hinged, is determined by the formula:

$$f = f_c\left[1 + k(1-\eta)\left(\frac{f_{st}}{f_c} - 1\right)\right] \tag{13}$$

where
k – Erection technology factor. k = 0,5 for propped beams; k = 0,3 for unpropped beams;
f_c – Sagging vertical deflection of composite beam with full connection, by the formula:

$$f_c = \frac{5qL^4}{384E_{st}I_{red}} \tag{14}$$

f_{st} – Sagging vertical deflection of steel beam without connection, by the formula:

$$f_{st} = \frac{5qL^4}{384E_{st}I_{st}} \tag{15}$$

where
q – Along load;
E_{st} – Modulus of elasticity of structural steel;
I_{red} – Second moment of area of the composite section;
I_{st} – Second moment of area of the structural steel section.

After the required amount of shear connectors has been determined, it is necessary to perform a cross-section check according to the limit states to determine the structure utilization. If the utilization is below 70%, it is recommended to reduce the cross section of the steel beam and recalculate the composite structure.

The approach for shear resistance of a connector P_{rd} depends on the type and fastening technology. For some welded simple-shape connectors there are different formulas exist, hoverer there the push-test is applicable for common case. Usually shear connectors resistance only for solid slabs is being defined. In case of profiled slabs, the P_{rd} should be multiplied on reduction factor k_t which could be calculated according to the SP 266.1325800:

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$$k_t = 0.7\frac{b_0(h_{an}-h)}{h^2\sqrt{n_r}} \leq 1.0 \tag{16}$$

where
b_0 – Width of haunch;
h_{an} – Overall nominal height of a connector;
h – Overall depth of the profiled steel sheeting excluding en1bosslnents;
n_r – Number of stud connectors in one rib.

Despites the existing k_t formula, SP 266.1325800 allows to use alternative approaches for k_t calculation. There were different approaches in the paper compared [9]. It was concluded that the k_t factor calculated according to the Konrad [14] provides more stable and less conservative results as (16) formula from SP 266.1325800 for specimen with local Russian profiled decks:

$$k_t = k_n\left(x_1\frac{h_{an}}{h} + x_2\frac{b_0}{h} + x_3\left(\frac{b_0}{h}\right)^2 + x_4\right) \tag{17}$$

where
x_1, x_2, x_3, x_4 - numeral factors according to the Table 1;
k_n – factor for 1 connector per rib is 1, for 2 and more connectors per rib is 0.8.

Table 1. Numeral factors for formula (17)

Connectors position		x_1	x_2	x_3	x_4
Favorable position	$h_{an}/h \leq 1.56$	0.24	0.145	0.03	0
	$h_{an}/h > 1.56$	0.318	0.103	0.003	0
Middle position	$h_{an}/h \leq 1.56$	0.25	0.17	$6.79 \cdot 10^{-4}$	0
	$h_{an}/h > 1.56$	$6.83 \cdot 10^{-4}$	0.042	$5.34 \cdot 10^{-4}$	0.663
Unfavorable position	$h_{an}/h \leq 1.56$	0.305	0.004	0.036	-0.095
	$h_{an}/h > 1.56$	0.026	0.266	0.029	0

As it was shown in the study [15], the impact of fire on a push-test specimen reduces the shear resistance of welded stud in proportion to the temperature increase. It is noteworthy that the failure mode of the connection depends on the type of reinforced concrete slab under consideration. The destruction of specimen with solid slabs occurred due to the welder collar at 600°C with 59% residual shear resistance. In profiled slabs, destruction occurred due to cracking of concrete with a decrease in the residual shear resistance 36%. It provoked an idea to use addition k_{temp} reduction factor to take the effect into account. The k_{temp} factor should be defined in push or full beam test for at least 20, 200, 350 and 500°C [9]. For X-HVB shear connectors it could be taken from ETA approval [16].

Table 2. k_{temp} values for X-HVB shear connectors

Design temperature on steel beam °C	**≤ 100**	**200**	**300**	**400**	**500**	**600**	**≥ 700**
k_{temp}	1.0	0.95	0.77	0.42	0.24	0.12	0

The initial data for k_{temp} reduction factor definition from the structure side should be a design temperature on steel beam. It could be defined with thermal calculation according to the STO ARSS 11251254.001-018-03 [17] (for uncoated steel structures) or with test data according to the GOST R 53295-2009 [18] (for PFP-coated steel structures). The last option assumes the test of default beam with PFP-coating, which provides data about temperature on the steel beam surface under coating by thermal indicators. The time-temperature diagram from the test report is used for calculation: postponing the value of the critical (design) temperature along the vertical axis, there is a point on the selected graph, that projects on the horizontal axis for showing the fire resistance limit of the structure.

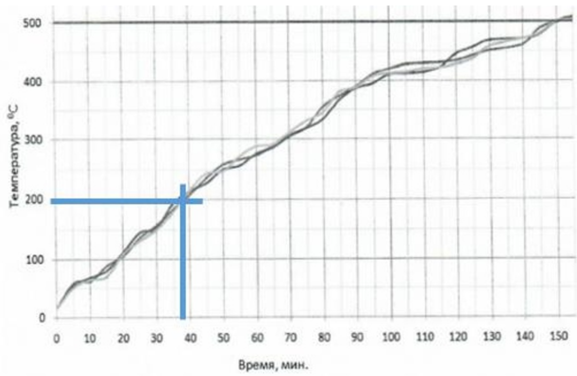

Fig. 2. Time-temperature diagram from the test report according to the GOST R 53295-2009

Thus, it is recommended to calculate the shear connector resistance according to the formula:

$$P_{rd} = P_{rd}^{`} \cdot k_t \cdot k_{temp} \tag{18}$$

where
$P_{rd}^{`}$ – Another requirement, that should me matched is;
k_t – reduction factor, calculated by (17);
k_{temp} – reduction factor, defined according to the Table 2.

3 Results

For the proposal approbation a working example has been chosen: the composite beam 9 meter span, slab with is 2 meter, live load is 1 ton/m^2. Slab is made with steel deck N75 (made according to GOST 24045-2016 [19]) with 180 mm thickness. Concrete grade is B25, steel reinforcement made of Ø10 mm bars, 200 mm spacing. The I-section type was predefined according to the SP 266.1325800 for the case of composite beam with full connection: 35B1 according to the GOST R 57837-2017 [20], steel grade is S245. Thus, the dead weight of 1 linear meter of the structure is 5.31 kPa, the linear load on the beam is 30.6 kN / m. As own fire resistance limit of the steel beam is less than 10 minutes, it is PFP-coating designed. The time-temperature diagram for the coating is shown on Fig 2.

The X-HVB 125 was chosen as a shear connector. The design resistance in solid slab is 30 kN according to the ETA. The working example sketch is shown on Fig 3.

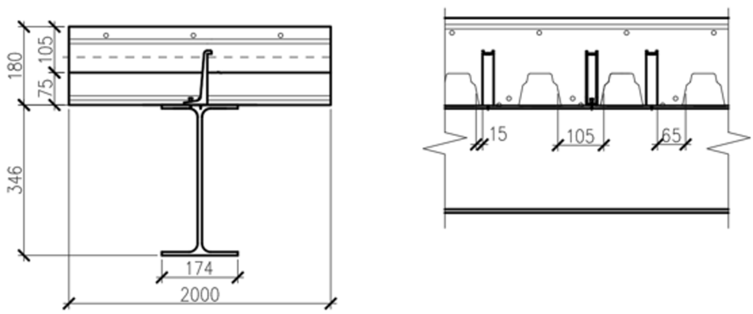

Fig. 3. The working example sketch.

The structure was calculated twice: with full connection according to the SP 266.1325800.2022 and with partial connection (the connection degree η = 0.51) according to new proposal.

Table 3. Result comparison

Parameter	SP 266.1325800	New proposal
Utilization	100%	87%
Number of shear connectors	62	48
Beam deflection	2,22 cm	3,58 cm

The structured designed according to new proposal is 13% less utilized with smaller quantity of shear connectors (22% less) under the same load. It provides benefits from the structure costs and labor time. The deflection of the composite beam designed according to new proposal is 38% higher, however doesn`t exceed the limit L/250 (3,6 cm).

4 Discussions

The proposal could provide less conservative results for composite beams with ductile shear connectors and at the same moment, it takes into account addition condition, such as decking geometry and performance of the structure under fire. The proposal is verified on powder-actuated shear connectors and should be double-checked with other working examples and connectors type.

The scope of new proposal should be clarified by additional calculation to highlight the cases, where it would be most effective to be used. There are also addition fire tests needed to clarify k_{temp} factors for different shear connector or develop a math approach to estimate the influence of fire on shear connectors resistance.

5 Conclusions

In conclusion, the paper discusses two approaches to shear connection design in composite floors: fully connected and partially connected. The new approach is preferred for civil structures and requires a ductile longitudinal shear behavior and a certain degree of shear connection. The study proposes an update to the SP 266.1325800 code, which is based on the same design principles as EN 1994-1 but does not include provisions for partial connections design. The proposed methodology includes a new set of provisions and is based on deep code analysis and comparison and previous authors' publications. The study also includes a working example that demonstrates the proposed methodology's benefits, such as a 22% reduction in the number of shear connectors and a 13% decrease in structure utilization. However, further research and testing are needed to verify the proposal's effectiveness for different types of shear connectors and working examples.

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