Mechanical interaction of a reinforced concrete slab and a steel beam in bridge spans

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Abstract. The paper aims to clarify the rules of calculation of steel-reinforced concrete bridge superstructures on the basis of generalization of modern scientific achievements in conjunction with the possibilities of computational software and computer technology in the field of calculations of multi-component sections without the need to use simplifying hypotheses that affect the accuracy of the calculation. An algorithm for calculating continuous multi-span beams of bridge structures with the possibility of taking into account the flexibility of the shear joint between concrete and steel structural elements is proposed, which increases the accuracy of determining the stress-strain state (SSS) of the structure. The results of the work can be used in the design of steel-concrete bridge superstructures. The significance of the work lies in the fact that the proposed analytical dependences in the joint use of modern computing power will clarify the calculation of steel-concrete bridges, thereby increasing the level of reliability of bridge structures, laid at the design stage.

1. Introduction

The purpose of this paper is to clarify the rules of calculation of steel-reinforced concrete spans of bridges to obtain analytical dependencies that take into account the shear stiffness between reinforced concrete and steel structural elements of bridges in single-span and continuous multi-span beams. Nowadays the solution of practical engineering problems is impossible without the appropriate computational software systems, so when obtaining analytical dependencies, the licensed LIRA-CAD software package [6] was used to model spans of steel-reinforced concrete bridges using the finite element method.

Currently, the calculation of steel-reinforced concrete structures in bridge building is regulated mainly by the following regulatory documents of the Russian Federation: CP (Code of Practice) 35.13330.2011 “Bridges and pipes. Updated edition of BC and R (Building Codes and Regulations) 2.05.03.84” and CP 159.1325800.2014 “Steel-reinforced concrete spans of road bridges. Rules of calculation.” At the same time, taking into account the effect of shear between reinforced concrete and steel structural elements of steel-reinforced concrete bridges is not considered in the current national standards. In CP 35.13330.2011 it is recommended: "9.4. The calculations should be carried out ... without taking into account the compliance of the joints of the steel and reinforced concrete parts".
But obtained by various authors experimental and theoretical studies carried out in this direction, [10, 14, 15], demonstrate significant differences in the stiffness values for the connecting elements of the steel and reinforced concrete parts of the structure cross section. Subsequent studies [1, 2, 8, 13] showed that the calculation of steel-reinforced concrete bridge spans under constant load requires taking into account the ultimate shear stiffness of the joining of a steel beam with a reinforced concrete slab. Moreover, under the action of a temporary load, the structure works as elastic with small shear displacements and significant shear stiffness of the join between the slab and the beam.

2. Materials and methods

As a justification for taking into account the flexibility of the joint of a steel beam with a reinforced concrete roadway slab, we consider the operation of a real object: a bridge across the Don River at 214 + 681 km of the R-298 Kursk – Voronezh highway (the part of R-22 “Kaspiy” highway in Voronezh region). The scheme of the bridge is 21+ (63x3) + 33x2, the central part is a 3-span continuous steel-reinforced concrete beam with a span of 3x63 m according to the model project 3.503-50 of “Lengiprotransmost” project office. The bridge was built in 1991, surveys were carried out in 2008 and 2013, during which the sagging of the first span of the steel-reinforced concrete part was noted. In 2008 the sagging was 136 mm, in 2013 it was 165 mm. Thus, over 5 years, sagging increased significantly, by 29 mm, in the absence of additional coating layers and other visible damage. This increase in sagging, the causes of which have not yet been established until the reconstruction, is interesting in terms of justifying the calculation of the compliance of the shear joint of a reinforced concrete slab with a steel beam.

As part of the bridge reconstruction in 2015-2016 a complete replacement of prefabricated reinforced concrete slabs of the carriageway with a monolithic slab was performed. During the slab dismantling, the unsatisfactory condition of the concrete was found in the filling of the first span prefabricated slabs in which the rigid supports were installed, which is a typical defect for this type of combination. These supports provide rigid adhesion of the reinforced concrete slab to the steel beam with high-quality concreting of prefabricated slab gaps [5, 7]. However, it was not possible to ascertain the condition of monolithic concrete in the gaps by non-destructive methods during the inspection process. The only indirect sign by which it was possible to assume their unsatisfactory condition was traces of leaks with leaching on the slabs lower surface.

Consider the carrying capacity assessment of the superstructure according to the results of calculating the cross section in the joint area with significant sagging, according to the classical technique and according to the approach proposed in this paper, taking into account the ductility of the joint of the steel and reinforced concrete parts.

2.1. Classical calculation technique

Calculation is carried out for the loads of A11 and H11 types in accordance with the CP 35.13330.2011.

Normal stresses in concrete at the center of gravity of the slab must satisfy the condition

$$\sigma_b = \frac{M_2^H}{n_b W_{6.0cm.6}} \leq m_b R_b.$$  

This condition is fulfilled as $\sigma_b = 6.53 \text{ MPa} < m_b R_b = 15.5 \text{ MPa}$.

Normal stresses at the low flange of the steel beam are calculated as:

$$\sigma_{l.f.} = \frac{M_2^{\max} - z_{bs} N_{br}}{W_{u,cm.}} + \frac{N_{br}}{A_s} \leq m \cdot R_y; N_{br} = A_b \cdot \sigma_b + A_r \cdot \sigma_r.$$

This condition is fulfilled as $\sigma_{l.f.} = 259.3 \text{ MPa} < m \cdot R_y = 295 \text{ MPa}$.

Normal stresses at the low flange of the steel beam are calculated as:
\[ \sigma_{u.f.} = \frac{M_{2}^{\max} - z_{bs} \cdot N_{br}}{W_{a.cm.}} - \frac{N_{br}}{A_{z}} \leq m_{2} \cdot m \cdot R_{y}. \]

This condition is fulfilled as \( \sigma_{u.f.} = 238.9 \text{ MPa} < m_{2} \cdot m \cdot R_{y} = 354 \text{ MPa} \).

In the calculations, the designations are used in accordance with the CP 35.13330.2011 where:
\( M_{2}^{U} = 28518 \text{kN} \cdot \text{m} \) is the bending moment resulting from the load; \( M_{2}^{\max} = 44370 \text{kN} \cdot \text{m} \) is the maximum bending moment; \( n_{b} = 6.0 \) is the reduction factor (according to the project); \( m_{b} = 1.2 \) is the coefficient accounting for the stresses that are balanced in the traverse joint section and that take place at the center of gravity of the concrete traverse section due to the concrete contraction and creep and because of the temperature changes; \( R_{b} = 13 \text{ MPa} \) is the calculated resistance of the concrete of the B25 type (M400) to the compression according to paragraph 7.24 of the CP 35.13330.2011; \( N_{br} = 8417 \text{ kN} \); \( \sigma_{l.f.} = 38 \text{ MPa} \); geometric characteristics of the cross-sections are adopted in accordance with the project: \( W_{a.cm.0} = 727517 \text{ cm}^{3} \); \( z_{bs} = 2.395 \text{ m} \); \( W_{a.cm} = 140200 \text{ cm}^{3} \); \( W_{a.cm} = 74300 \text{ cm}^{3} \); \( A_{z} = 970 \text{ cm}^{2} \); \( A_{b} = 12602 \text{ cm}^{2} \); \( A_{l} = 48 \text{ cm}^{2} \). Coefficient \( m_{2} \) is introduced to account for the unloading effect that the plate has on the upper flange. In the present case \( \sigma_{l.f.} < 0.8 \cdot R_{b} \), and that means that \( m_{2} = 1.2 \).

All checks have been performed, the superstructure corresponds to the load classes A11 and H11. It should be noted that in other characteristic sections (above the supports and in the middle of the second span), all tests performed on the basis of the hypothesis of flat sections were satisfied.

When calculating numerically in the LIRA-SAPR software package according to the finite element model with an unyielding connection of a reinforced concrete slab with a steel beam and the same loads for normal stresses in the upper flange of the steel beam, \( \sigma_{u.f.} = 260 \text{ MPa} \), in the upper flange - \( \sigma_{u.f.} = 235.8 \text{ MPa} \). The results are in good agreement with those presented above.

2.2. Numerical calculation with yielding connection

We compose the calculation model in the LIRA-CAD software package, based on the following assumption. If from 2008 to 2013 there was an increase in the sagging span of 29 mm, but the condition of the joints of steel beams is good and indicates an initial construction rise; lengthy processes in a reinforced concrete slab (shrinkage and creep) should have been completed long before the beginning of the sagging growth; since additional coating layers were not laid in the considered period, the only reason why the sagging can increase in a rather short period of time is a decrease in the shear stiffness of the joint section. This is precisely what the further dismantling of the slabs during reconstruction showed, due to the degradation of concrete in the supports.

The inverse problem was solved to quantify the reduction in shear stiffness of the supports: by the selection of shear rigidity in finite element models, it appears that the increase in deflection from permanent loads of 29 mm occurs at lower linear shear stiffness of the joint to 9810 \text{kN/m}^{2} (though for hard supports in perfect condition, the value should be 3 orders of magnitude higher, about 1275000 \text{kN/m}^{2}). Thus, it is possible to draw a conclusion about almost complete violation of the stops in this case. The load capacity of the structure with such shear stiffness will be reduced.

Figure 1 represents the numerical calculation of the normal stresses isofield in a steel beam (the upper shelf is at the bottom so that the stress scale goes from negative to positive values) with a joint stiffness of 9810 \text{kN/m}^{2}: a) only from constant loads, b) from constant in combination with A11 (traffic load).

Taking into account the coefficient \( m_{2} = 1.2 \), which includes the unloading effect of the slab on the upper flange (as was assumed in the calculation according to the classical technique), the ultimate stress for the upper flange from constant loads is 303.9 \text{MPa}, and it does not exceed the maximum allowable 354 \text{MPa}. But the maximum tension from constant loads in combination with A11 is 415.8 \text{MPa}, and it already exceeds the maximum permissible value. Based on the increasing sagging, if we take the shear stiffness of the joint 9810 \text{kN/m}^{2}, then the span carrying capacity is unsecured, and the permissible load class is \([(354-303.9)/(415.8-303.9)] \cdot 11 = 0.44 \cdot 11 = 4.8 \), which is significantly lower than the standard value. Thus, the calculation of steel-reinforced concrete bridge spans, taking into account the shear...
stiffness of the connecting joint, gives a load capacity that is significantly different from that obtained using the classical technique. This difference is especially noticeable for operating facilities in which the connection of concrete with a steel beam can be severely disrupted relative to design values.

![Figure 1. The normal stresses isofields in a steel beam.](image)

It should be noted that the theoretical value of the slap shear along the beam at the specified joint stiffness of 9810 kN/m² is 4.4 mm at the end, and only 0.34 mm in the considered section. Thus, even if shear measurement was part of the survey program, such values can hardly be detected, especially in the case of damage to the concrete slab in the area of the expansion joint. The given example clearly shows the inconsistency of the definition of clause 6.6.1.1 of the Eurocode [11, 12] for an unyielding joint, which sets the value of the maximum shift at a level of 6 mm, less than which the joint should be considered unstable.

We obtain the analytical dependences of the slip of a reinforced concrete slab over a steel beam on the shear stiffness value for single-span and multi-span continuous beams.

For a single-span steel-reinforced concrete beam, the design scheme, modeling the shear stiffness of the connecting elements, and the results of numerical calculations are presented in [4]. Numerical experiments using LIRA-CAD allowed us to obtain graphical dependences of normal compressive stresses in the upper flange of a steel beam and in a reinforced concrete slab, normal tensile stresses in the lower flange of a steel beam, deflection of the beam, and shear at the end of the beam, depending on the linear shear stiffness of the joint. The range of shear stiffness considered in this case was chosen based on experimental values for flexible rod supports, as the most common in practice of real designing structures for combining reinforced concrete slabs with steel beams presented in the dissertation [9]. An analysis of the effect of shear stiffness on the assembly’s stress and strain state showed that even with small slippage values of up to 1 - 2 millimeters along the edges of the beam, the
forces in the reinforced concrete slab decrease, and in the upper flange of the steel I-beam increase, redistribution of forces occurs. Therefore, in the practice of real steel-reinforced concrete bridge spans designing, it is necessary to take into account the shear stiffness between the steel beam and the concrete slab of the roadway.

The bonds connecting the reinforced concrete slab and the steel beam can be either continuously distributed along the joint length, or concentrated at separate points of the composite structure length (discrete). In the calculations of steel-reinforced concrete beams, it is recommended to take the relationship between the strains that arise inside the bonds of the composite beam and the internal forces caused by these strains as linear:

$$\delta_x = \frac{S_{h,x} \cdot k}{\varsigma}, \quad (1)$$

where $\delta_x$ is the deformation of the mutual shift of the layers connected by the shift ties, $(m)$; $S_{h,x}$ is the moving force effecting one tie, $(\kappa N)$; $k$ is the number of the ties per joint length unit, $(1/m)$; $\varsigma$ is the coefficient of stiffness for the shift ties a, $(\kappa N/m^2)$.

Consider a hinge-supported beam with a span $l$ loaded with a uniformly distributed load along the entire length $q$. In an arbitrary section $x$ from the reference section, the transverse force is equal to:

$$Q = \frac{q \cdot l}{2} - q \cdot x = q \cdot \frac{l - 2x}{2}. \quad (2)$$

With an absolutely rigid shear joint ($\varsigma \to \infty$), the displacement $\delta_x = 0$, and the tangential stresses at the joint level are determined by the Zhuravsky formula:

$$\tau = \frac{Q \cdot S_{red}}{I_{red} \cdot b_s} = \frac{q(l - 2x) \cdot S_{red}}{2 \cdot I_{red} \cdot b_s}, \quad (3)$$

where $S_{red}$ is the static moment of the section of the steel beam cut off at the seam level, $(m^3)$; $I_{red}$ - moment of inertia of the full steel-reinforced concrete section, $(m^4)$; $b_s$ is the width of the joint $(m)$, in this case taken equal to the width of the steel beam upper flange (as is the case in most steel-reinforced concrete structures).

During composite structure operation, forces arise in the shear bonds, which are functions of the $x$ coordinate measured along the length. The value of these efforts, referred to the unit of the joint length, is denoted by $S_x (S_x = S_{h,x} \cdot k, \kappa N/m)$. The magnitude of the shear forces $S_x$ acting on a unit of the joint length for an absolutely rigid joint will be equal to:

$$S_x = \tau \cdot b_s = \frac{q(l - 2x) \cdot S_{red}}{2 \cdot I_{red}}. \quad (4)$$

Substituting (4) in (2), we obtain the relationship between the displacement $\delta_x$ and the shear stiffness of the joint $\varsigma$:

$$\delta_x = \frac{S_{h,x} \cdot k}{\varsigma} = f \left[ \frac{q(l - 2x) \cdot S_{red}}{2 \cdot I_{red} \cdot \varsigma} \right]. \quad (5)$$

The function $f$ should be determined experimentally and cannot be considered linear with respect to $\varsigma$, since this expression is obtained under the assumption that, regardless of the value, the influence of the shear stress (3) for a flat section (at $\varsigma \to \infty$) on the real shear stress, which depends on the stiffness of the joint, remains.
The shear stress value \( S_x \), as well as the shear value \( \delta_x \), depends on the shear stiffness of the joint. When \( \zeta = 0 \): \( S_x = 0 \), \( \delta_x \) in the middle of the span is zero, and in the reference section of the articulated beam under the action of a uniformly distributed load, it has a maximum value determined by the formula

\[
\delta_x = Z_{b,s} \cdot \frac{16f_q}{5l},
\]

where \( Z_{b,s} \) is the distance between the centers of gravity of the concrete and steel sections; \( f_q \) is the elastic deflection in the middle of the span of an articulated beam; \( l \) is the calculated span of the beam.

When \( \zeta = 0 \): \( \delta_x = 0 \) (there is no displacement, the hypothesis of plane sections without shifting the components along the entire length of the span is fulfilled), \( S_x \) has a maximum value and is determined by the formula (4).

Numerical experiments have established that regardless of the span length, geometric parameters of the rods and the value of the uniformly distributed load constituent, for the values of \( \zeta \geq 200000 \) kN/m², the dependence for a single-span beam has the form:

\[
\delta_x(x, \zeta) = 0.36 \cdot \left[ \frac{q \cdot (l - 2x) \cdot S_{red}}{2 \cdot I_{red} \cdot \zeta} \right]^{0.87}.
\]

The coincidence of the values obtained using a numerical experiment and calculated from the analytical dependence (7) is in the error range of not more than 5%. This is due to the fact that in order to derive dependence (7), the criterion of the ultimate shear stress in a flat section (4) which has an effect just at rather large values of shear stiffness, was chosen. Graphic dependences are created for beams 42, 63, 84 and 126 m long [3].

![Figure 2. The scheme of continuous 3-span beam with characteristic bending sections moments.](image)

By the numerical experiments similar to those performed for a single-span beam, it was found that for multi-span beams (Figure 2), regardless of the spans length, their number, the ratio of lengths, geometric parameters of the constituent elements (both common for the whole continuous beam and different in spans), the position of the section under consideration (in the edge or middle span) and the shear forces, for \( \zeta \geq 200000 \) kN/m², the dependences between the displacement \( \delta_x \) and the shear stiffness of the joint have the form:

\[
\delta_x(x, \zeta) = 0.76 \cdot \left[ \frac{q \cdot (l - 2x) \cdot S_{red}}{2 \cdot I_{red} \cdot \zeta} \right]^{0.97}.
\]

\[
\delta_x(x, \zeta) = 0.18 \cdot \left[ \frac{q \cdot (l - 2x) \cdot S_{red}}{2 \cdot I_{red} \cdot \zeta} \right]^{0.75}.
\]
In this case, for the sections I with positive moments (Figure 2) the formula (8) should be used, for the sections II with negative moments the formula (9) should be used.

Figures 3 - 5 present obtained by numerical experiment (blue curves) and analytically by the formula (8) and (9) (brown curves) graphical dependences of the shear displacement $\delta_x$, (ordinate axis, mm) on the shear stiffness of the joint $\zeta$, (abscissa axis, kN/m) for the 3-span beam.

**Figure 3.** The comparison of the numerical (blue curve) and analytical (8) (brown curve) dependencies in the initial section of the first span of the continuous beam according to the scheme 63 + 84 + 63 m, positive moment ($x = 10 m$: span 1, the beginning).

**Figure 4.** The comparison of the numerical (blue curve) and analytical (8) (brown curve) dependencies for the average span of a continuous beam according to the scheme 42 + 63 + 42 m, positive moment ($x = 70 m$: span 2, the middle part).
Figure 5. The comparison of the numerical (blue curve) and analytical (9) (brown curve) dependencies near the support in a continuous beam according to the scheme 63 + 84 + 63 m, negative moment ($x = 155$ m: span 3, next to the support).

3. Results and discussion
Thus, for hinge-supported beams, the calculation of the dependence of the absolute value of slap slippage on the beam from the value of shear stiffness $\delta_x = f(x, \xi)$ is calculated by three formulas depending on the static scheme of the beam and the calculated section’s position (Table).

| Functional dependence $\delta_x = f(x, \xi)$ |
|---------------------------------------------|
| Single span beam                             |
| $\delta_x = 0.36 \cdot \left[ \frac{q \cdot (l - 2x) \cdot S_{red}}{2 \cdot I_{red} \cdot \xi} \right]^{0.87}$ |
| Multi-span beam                              |
| positive bending moment (Sections I, Figure 2) $\delta_x = 0.76 \cdot \left[ \frac{q \cdot (l - 2x) \cdot S_{red}}{2 \cdot I_{red} \cdot \xi} \right]^{0.97}$ |
| negative bending moment (Sections II, Figure 2) $\delta_x = 0.18 \cdot \left[ \frac{q \cdot (l - 2x) \cdot S_{red}}{2 \cdot I_{red} \cdot \xi} \right]^{0.75}$ |

In [3] integral equalities, which for a single-span beam make it possible to determine the upper limit of shear stiffness at which it is permissible to calculate the span of the bridge using the classical model of flat sections without taking into account the shift between the reinforced concrete slab and the steel beam, are presented. The upper limit of shear stiffness for an arbitrary section $x$ from the beginning of a continuous multi-span beam can be determined using the generalized integral equation for $N_C$, the total shear force in the joint, accumulated along the structure length from the beginning to the section under consideration:
$$N_\xi = \int_{x_1}^{x_2} \xi \cdot m_1 \left[ \frac{Q_{eq,1}(x) \cdot S_{red}}{I_{red} \cdot \xi} \right]_{x_1}^{x_2} dx + \int_{x_1}^{x_2} \xi \cdot m_2 \left[ \frac{Q_{eq,2}(x) \cdot S_{red}}{I_{red} \cdot \xi} \right]_{x_1}^{x_2} dx + \int_{x_3}^{x_4} \xi \cdot m_3 \left[ \frac{Q_{eq,3}(x) \cdot S_{red}}{I_{red} \cdot \xi} \right]_{x_3}^{x_4} dx + \ldots$$

In the equation (10):
section $(0;x_1)$ is the edge section with a positive bending moment in the first span of the continuous beam according to Fig. 2, $(x_i;x_j)$ is the section in the first span of the continuous beam with a negative moment adjacent to the first intermediate support, $(x_j;x_i)$ is the section in the second span of the continuous beam with a negative moment adjacent to the first intermediate support, $(x_i;x_j)$ is the section in the second span with a positive bending moment, $(x_i;x_j)$ - the portion in the second span of the continuous beam with a negative moment adjacent to the second intermediate support, $(x_j;x_i)$ is the section in the third span of the continuous beam with negative moment, adjacent to the second intermediate internal support and so on until the section in which the considered section takes place;

$q_{eq,i}(x), \ldots, q_{eq,6}(x)$ are the values of the equivalent transverse force $Q$ from the distributed load $q$ in section $x$ from the beginning of the beam calculated for each section;

$m_1 = 0.76, m_2 = 0.97$ are the constants in formula (8) for sections I with positive moments (Fig. 2);
$m_2 = 0.18, m_3 = 0.75$ are the constants in formula (9) for sections II with negative moments (Fig. 2).

In this case, the limit value $N_\xi$ for continuous multi-span beams is recommended to be determined according to the notes to the tables 9.4 - 9.6 “CP 35.13330.2011. Bridges and pipes. Updated version of BC and R 2.05.03.84”.

4. Conclusion

So, during numerical experiments, it was found that in the presence of a shear in the connecting joint between the steel beam and the reinforced concrete slab of the steel-reinforced concrete spans of the bridge structures, a significant redistribution of stresses between the beam and the slab occurs. Therefore, when designing steel-reinforced concrete bridge spans, it is necessary to take into account the shear stiffness between the steel beam and the reinforced concrete slab of the roadway, which is confirmed by a practical calculation example for a real object. For single-span and continuous multi-span hinge-supported beams under the action of a load uniformly distributed over the entire length of the load, based on numerical experiments, new analytical formulas for the displacement of a reinforced concrete slab along a steel beam depending on the shear stiffness of the joint are obtained. In the real designing practice, their application allows to obtain more accurate stress distribution fields in composite structures of steel-reinforced concrete span structures of bridges.

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