Verification of the Behavior of Deck Bridges with Encased Filler Beams

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Abstract— This paper pertains the experimental research into filler-beam deck bridges with encased beams of various sections.For more effective steel utilization inverted T-sections are used. Resistances of deck bridges using various types of composite action between steel members/filler beams and concrete are compared. Design values of resistance moment for individual types of beam are calculated according to Eurocode 4. The theoretical calculations and their results are further verified in experiments.

Keywords—deck bridges, beam, composite action, resistance moment.

I. Introduction

LOAD-bearing structures of deck railway bridges with encased filler beams have been used for short and middle spans of a maximum of 24 metres. For over a hundred years they have been designed in cases with little headroom. The first bridges were constructed with no interaction between steel beams and concrete floor slabs, the structural steel working as a bearing element and the concrete in the structure as a hardening and filling element. Later, in the second half of the 20th century, more developed bridge designs were introduced where encased steel beams were used acting compositely with a concrete floor slab – the concrete transmitting actions in compression and the steel acting in tension. These structural designs were based on the method of permissible stresses and have been in use up to present. According to [6] it can be assumed that bridges designed employing this methodology meet the requirements stipulated in the current technical standards.

Filler-beam deck bridges with encased steel beams are more and more commonly used nowadays, especially in construction and reconstruction of railways. In reconstructions, they are mainly used in replacement of bridges with direct railway bedding that do not comply with relevant standards in terms of the required velocities and operational aspects.

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Basic rules and requirements set for the design of filler-beam deck bridges are provided in STN EN 1994-2, a European standard that specifies some common structural design and verification rules for sections based either on the plastic theory (providing that cross-sections are classified as Class 1 or 2 according to the above-mentioned standard) or on the elastic theory. According to the specified standard, mechanical shear connection need not be provided.

Rolled or welded I-sections have been used in the majority of recently designed and built filler-beam deck bridges. The research programme in progress at the Institute of Structural Engineering at the Civil Engineering Faculty of the Technical University in Košice pertains to the theoretical and experimental verification of such bridges but with encased modified steel sections (filler beam decks) designed to take advantage of the interaction of a concrete floor slab with steel sections, thereby considerably saving the consumption of steel.

II. Test Specimen

The sections and dimensions of test specimens were designed to meet the structural requirements placed on this type of bridge and allow experimental testing in the setting of the laboratories of the Civil Engineering Faculty. The experiments were carried out in three variants and the specimens were designed so that the equal resistance was reached in each of the variants.

The first variant of specimens marked as SPC is made from encased rolled IPE-200 sections. The concrete cover above the steel beam is 70 mm. The overall depth of the deck is 270 mm and the width is 670 mm, which corresponds to the axial distance of steel beams in deck bridges. There are three concrete reinforcing bars 12 mm in diameter at the upper edge of the deck. Transverse reinforcement consists of stirrups 12 mm in diameter placed in an axial distance of 300 mm. The length of specimens is 3000 mm. Both cross-section and longitudinal section are shown in Fig. 1.

The second variant of specimens marked as SPP is made from encased rolled IPE-220 sections cut half longitudinally so that there are two T-sections made. The concrete cover above the beam is 160 mm. The dimensions of the specimens and reinforcement bars are identical to those in variant one. Again, the cross-section and longitudinal section are shown in Fig. 2.

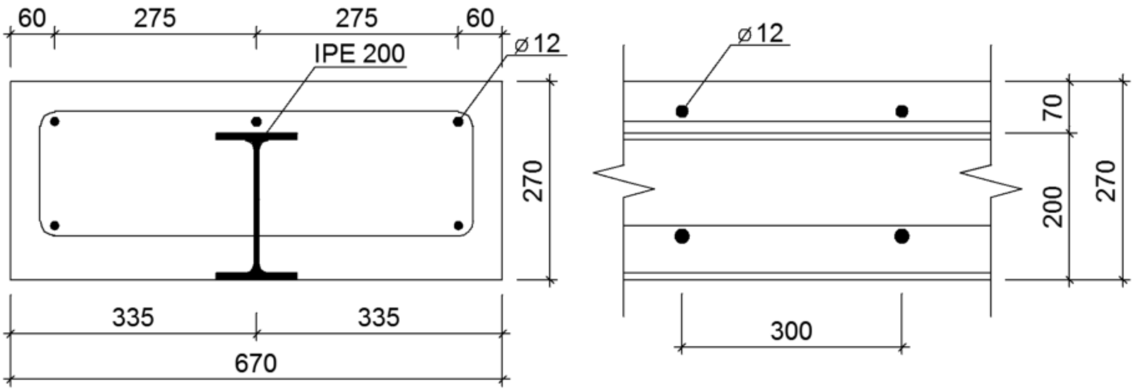

Fig. 1 Cross section and longitudinal section of SPC specimen

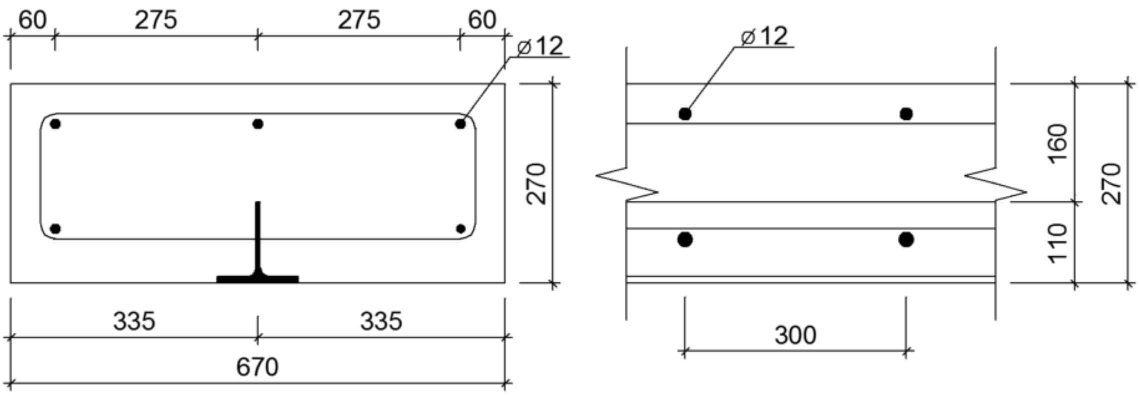

Fig. 2 Cross section and longitudinal section of SPP specimen

The third variant of specimens marked as SPH is identical to the SPP variant from encased rolled IPE-220 sections cut longitudinally into two T-sections. However, cutting the beam straight in the former case is replaced by cutting it in a comb-like manner creating thus the comb-like edge of the section in the latter case. There is a reinforcemnet bar in each tooth of the comb-like edge. The concrete cover above the beam is 131.5 mm. The dimensions of the specimens are also identical. The cross-section and the longitudinal section are shown in Fig. 3.

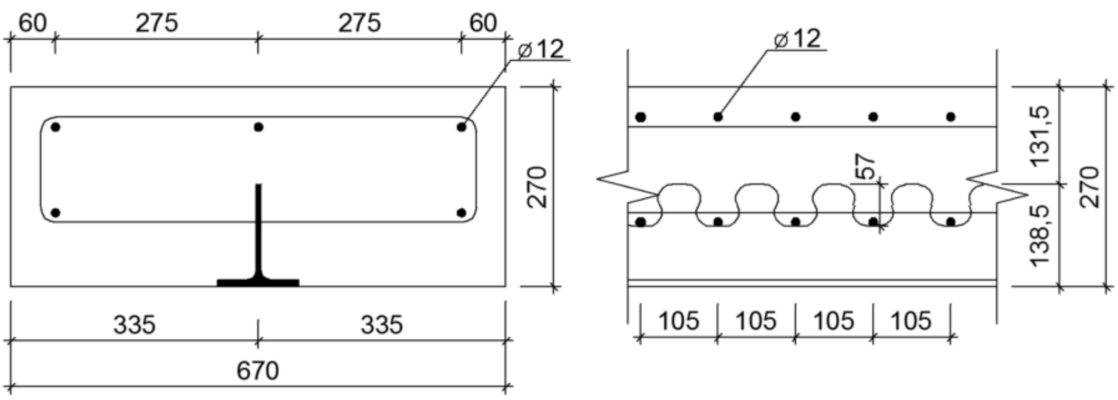

Fig. 3 Cross section and longitudinal section of SPH specimen

III. EXPERIMENT

Both the preparations of specimens and tests were conducted in the laboratories of the Institute of Structural Engineering. Three specimens of each variant were tested. They were placed on steel support beams resting on both ends. At one end the bearing was fixed in both horizontal and vertical directions, while the other end could move horizontally. Both bearings were hinged (Fig. 4).

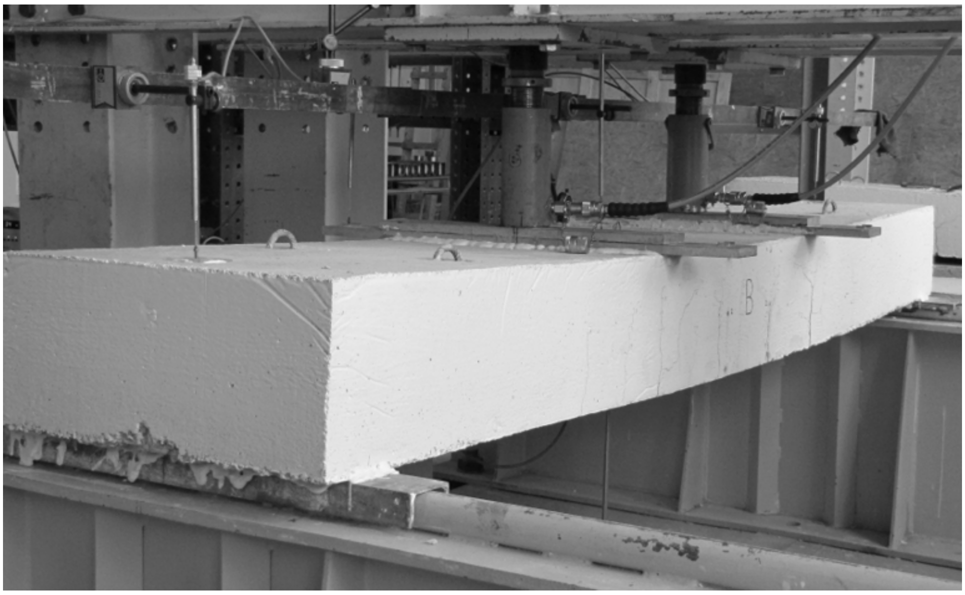

Fig. 4 Specimen during experiment

Cylinders 70 mm in diameter used to impose load on the specimens were placed on bearing plates/mats and then on the beam while leaning against the steel loading frame.

The load was imposed gradually in steps by 7.5 kN per each cylinder. The specimens were unloaded twice: the first time from a load of 60 kN to 15 kN and the second time from 105 kN to 30 kN. During the loading phase, when the load reached the value of 15 kN and thus the concrete exceeded its ultimate tensile strength, hair cracking occurred in the tension field of concrete. Later, the cracks opened up and increased until they were approximately 200 mm long in the SPC variant of specimens and 230 mm in the SPP variant of specimens, which was as deep as to the anticipated position of plastic neutral axes in both cases.

The tests were finished when the load could not be increased any longer as the deflections rose rapidly and uncontrollably.

IV. CALCULATION

For the determination of flexural stiffness of a composite cross-section and the calculation of its elastic bending resistance, it is first necessary to calculate the second moment of area of an ideal section. This is the characteristic that takes account of different structural properties of concrete and steel.

Cracks appear in the concrete part of the section where tensile actions exceeds the tensile strength of concrete. When determining the flexural stiffness, it is necessary to eliminate the cracked part of the section from the calculations. The tensile strength of concrete is rather low and the un-cracked concrete in tension is situated only near the centre of gravity, thus its contribution to the amount of flexural stiffness can be neglected. Therefore, concrete acting in tension is not considered in the calculations. The position of the centre of gravity in an ideal section is not uniquely defined. It can be determined iteratively from the following recurrent formula:

$$z_{i,(N+1)} = \frac{A_a.(h - z_a) + \dfrac{b.z_{i,(N)}^2}{2.n_0}}{A_a + \dfrac{b.z_{i,(N)}}{n_0}} \tag{1}$$

where:

z_i – the position of the centre of gravity of an ideal section taken from the upper edge of the section
A_a – the cross-sectional area of the structural steel section
h – the overall depth of a steel section
b – the width of a steel section

The iterative calculations finish in the condition given as $\left| z_{i,(N)} - z_{i,(N+1)} \right| \le 0{,}001$ and then $z_i = z_{i,(N+1)}$. The ratio of modulus of elasticity is determined by the modular ratio $n_0 = \frac{E_a}{E_c} = 6{,}774$.

The modulus of elasticity considered are:
E_a = 210 000 MPa for structural steel and
E_c = 31 000 MPa for concrete.

The second moment of area of an ideal cross-section I_i is consequently determined as:

$$I_i = I_a + A_a.(h - z_i - z_a)^2 + \frac{b.z_i^3}{n_0.3} \tag{2}$$

where:
I_a – the second moment of area of a structural steel section to its own axis
z_a – the position of the centre of gravity of a steel section taken from the lower edge of the section

In the calculations of elastic and plastic bending resistance, the following material characteristics obtained experimentally were used:

f_{yf} = 293 MPa – the nominal value of the yield strength of the structural steel of flanges
f_{yw} = 254 MPa – the nominal value of the yield strength of the structural steel of webs
f_{ck} = 25 MPa – the characteristic value of the cylinder compressive strength of concrete at 28 days

Elastic bending resistance is given by the moment that causes stresses in a section that are equal or lower than the yield strength of structural steel and/or the compressive strength of concrete. Based on this condition the elastic resistance moment $M_{el,cal}$ was calculated. Its values for the individual specimen variants are shown in Table 1.

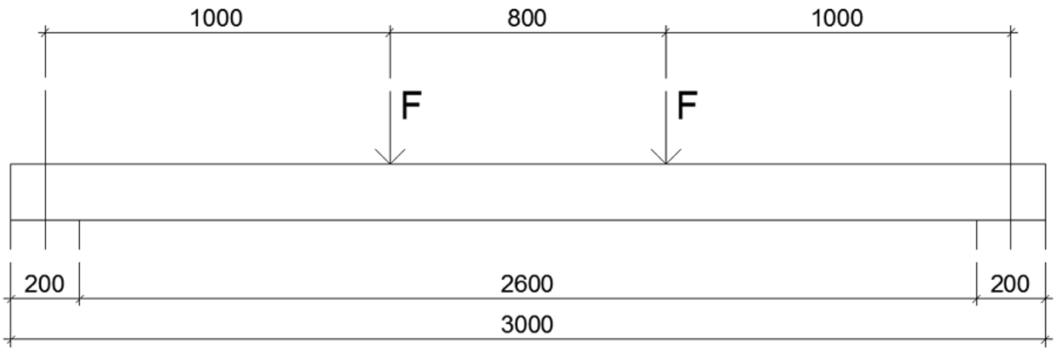

Fig. 5 Loading arrangements during the experiment

Based on the loading arrangements given in Fig. 5, the mid-span deflection is determined by the formula:

$$\delta = \frac{F_{pl,cal}.x}{24.E_a I_i}\left(3.l^2 - 4.x^2\right) \tag{3}$$

where:
l – the specimen span
x – the distance of the force from the nearest support

For deck bridges with encased filler-beams Eurocode 1994-2 recommends considering the flexural stiffness given by the average stiffness value of the un-cracked section and the section with the maximum crack length. However, the calculations proved that it is more accurate to use the stiffness of the cracked section for the whole specimen. This is due to the fact that the specimens were loaded by two identical forces. For such types of loading the value of maximum bending moment is not reached in one point, but it is constant along the whole area between the loading cylinders. The length of this area is approximately one third of the total specimen length and it is situated around the centre of the specimen. The calculated values of deflection are presented in Table 1.

Plastic bending resistance is determined assuming the equality of tensile and compressive forces while reaching the ultimate yield strength across the whole steel section and the stress in concrete corresponding to 0.85xf_{ck}. The plastic resistance moment $M_{pl,cal}$ calculated for the individual specimen variants is also in Table 2. The shape of plastic stress in the cross-section of the SPC specimens is shown in Fig. 6, of the SPC specimens in Fig. 7 and of the SPH specimens in Fig. 8.

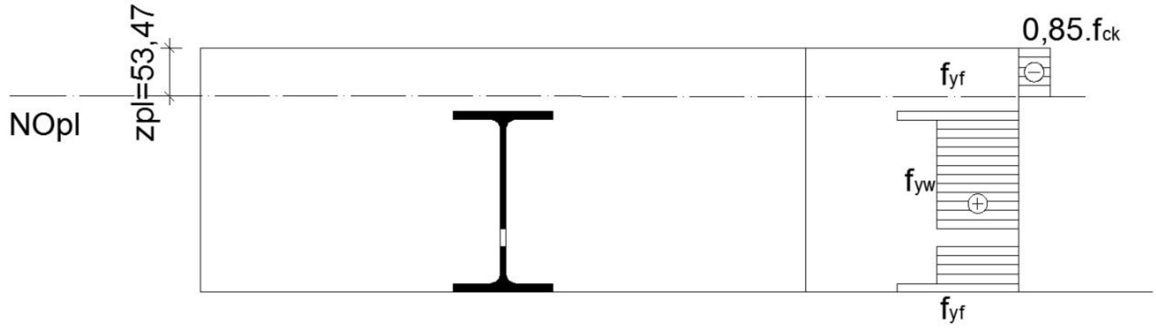

Fig. 6 Plastic stress in SPC specimen

The neutral axis is situated near the upper flange of the steel beam, so its contribution to the bending resistance is minimal.

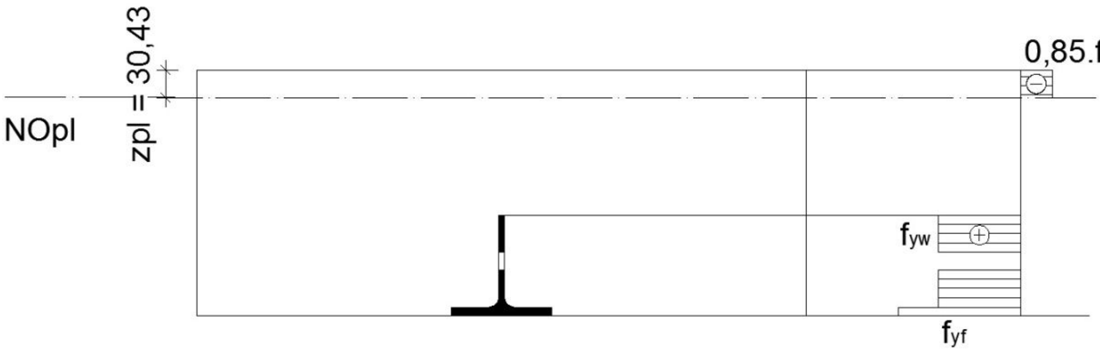

Fig. 7 Plastic stress in SPP specimen

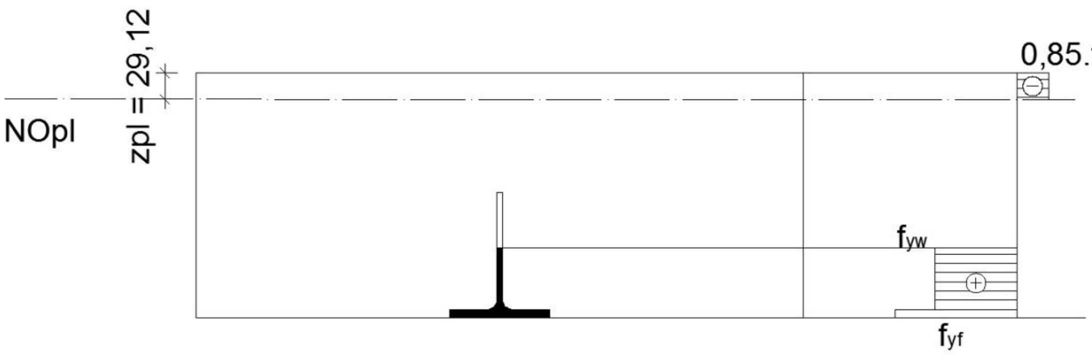

Fig. 8 Plastic stress in SPH specimen

V. Test Results

All variables were observed and recorded continuously and their average values evaluated graphically. A correlation between the overall mid-span deflection and the loads imposed is shown in Fig. 9. Similarly, the steel strain-load and concrete strain-load correlations are given in Fig. 10 and Fig. 11.

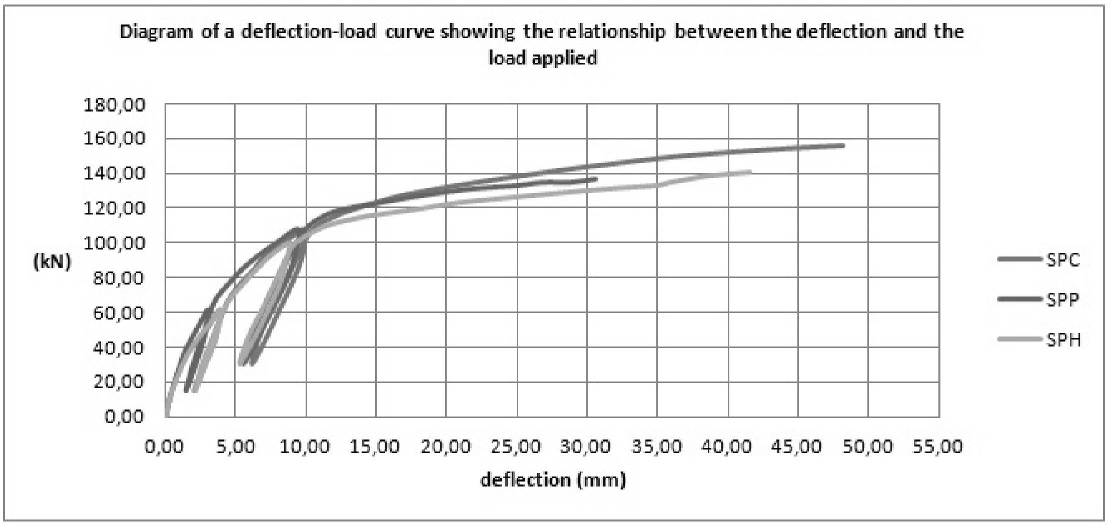

Fig. 9 Diagram of a deflection-load curve showing the relationship between the deflection and the load applied.

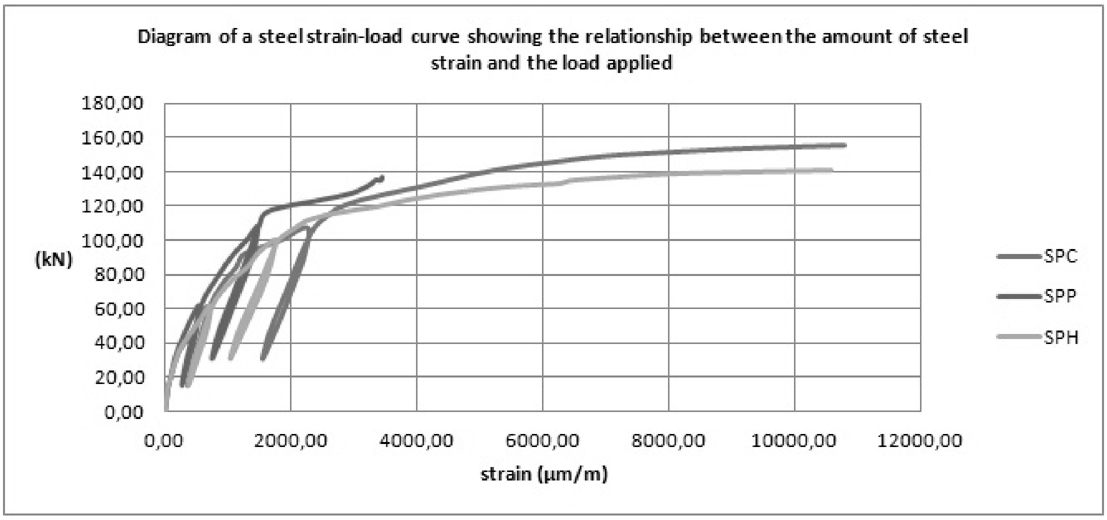

Fig. 10 Diagram of a steel strain-load curve.

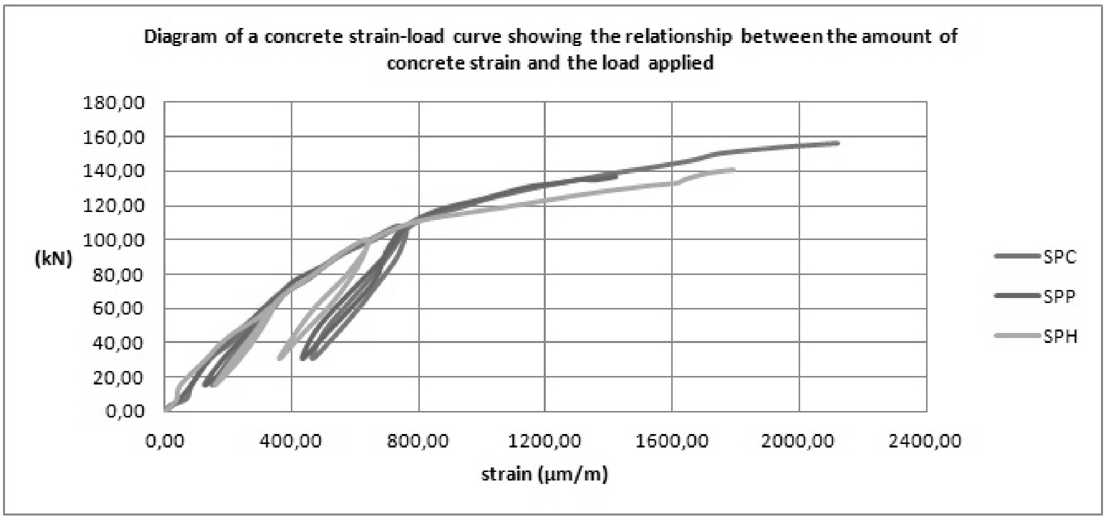

Fig. 11 Diagram of a concrete strain-load curve.

Table 1 compares the values of deflection $\delta_{el,exp}$ detected by experiment and those $\delta_{el,cal}$ calculated for the load corresponding to the state of reaching the elastic bending resistance.The same table also presents the calculated values of the elastic resistance moment $M_{el,cal}$.

Table 1: Calculated and measured values of elastic moment and deflection and their comparisons

Spec.	$M_{el,cal}$ (kNm)	$F_{el,cal}$ (kN)	$\delta_{el,cal}$ (mm)	$\delta_{el,exp}$ (mm)	$\delta_{el,av}$ (mm)	Diff. (%)
SPC - 1				4,92		
SPC - 2	76,67	72,36	4,92	4,95	4,93	+0,2%
SPC - 3				4,92		
SPP - 1				4,06		
SPP - 2	84,20	79,98	4,99	4,05	4,06*	-18,6%*
SPP - 3				4,07		
SPH - 1				5,05		
SPH - 2	84,16	79,95	5,00	5,07	5,06	+1,2%
SPH - 3				5,06		

$M_{el,cal}$ – calculated elastic resistance moment
$F_{el,cal}$ – force causing $M_{el,cal}$
$\delta_{el,cal}$ – calculated deflection by $F_{el,cal}$
$\delta_{el,exp}$ – measured deflection by $F_{el,cal}$
$\delta_{el,av}$ – average deflection by $F_{el,cal}$

* in these specimens the effect of slip occurred in composite sections, therefore, the initiation of cracks and their further development was different from the other specimens

In the elastic stage of experiment, the measured deflections corresponded to the assumptions made in the structural analysis in the case of the specimen variants with I-sections and T-sections with comb-like edges. The measurements of relative deformations of structural steel and concrete showed that the effect of slip between the concrete and the steel sections was observed in the variant with T-sections with straight edges. Composite action provided only by the mechanical bond/cohesion forces of both materials was insufficient. As a result, the values of deflection measured in this specimen variant differed from the calculations.

The values of maximum forces were measured in the situation when noticeable yield in the steel occurred. More detailed information on the resistance of the specimens is provided by the value of the moment measured just before the occurrence of this phenomenon. It happened under the load that caused a dramatic increase in deformation. The values of the corresponding forces $F_{pl,exp}$ for the individual specimens are shown in Table 2. In the Table are the measured values of resistance moments $M_{pl,exp}$ and their average values for the individual variants. The experimentally observed resistance moments were then compared with the calculated $M_{,pl,cal.}$

Table 2: Calculated and measured values of plastic moment and their comparisons

Spec.	$M_{pl,cal}$ (kNm)	$M_{pl,exp}$ (kNm)	$M_{pl,av}$ (kNm)	Diff. (%)
SPC - 1	107,93	127,4	124,8	+15%
SPC - 2		123,5		
SPC - 3		123,5		
SPP - 1	101,64	119,5	122,1	+20%
SPP - 2		123,4		
SPP - 3		123,4		
SPH - 1	99,91	115,8	117,0	+17%
SPH - 2		115,8		
SPH - 3		119,5		

$M_{pl,cal}$ – calculated plastic resistance moment
$M_{pl,exp}$ – measured plastic resistance moment
$M_{pl,av}$ – average plastic resistance moment

From the above analysis it can be concluded that the ultimate resistance moments in the laboratory conditions were in excess of those assumed in the calculations in all cases. A reserve of bending resistance that remained was 15% for the variant with encased I–sections; however, it was 20% for straight-edged T–sections and 17% for those with the comb-like edge. Furthermore, it is clear from the correlations shown above that the maximum strains in the specimens with the straight-edged T–sections were lower than those in the specimens with encased I–section beams. In the specimens with the comb-like-edged T-sections the maximum strains were approximately as high as in the specimens with I-sections. That was due to the fact that the effect of slip at the interface between the concrete and the steel beam that occurred in the specimens with the straight-edged T–section beams was greater than that in the case with I-section beams.

VI. Conclusion

The anticipated values of ultimate bending moments in the individual variants of deck bridges were reached in all experiments. The results confirm that the utilization of T-sections in filler beams is an appropriate solution from the point of view of their bending resistance. Nevertheless, the composite action between steel and concrete in the deck structures using straight-edged T-sections as filler beams remains problematic as the bonding between the two materials is not sufficient. However, as can be seen from the above experiments, the T-sections cut longitudinally in a comb-like manner can provide sufficient composite action between the materials. As a result, the research into filler-beam deck bridges with encased T–section beams at the Institute of Structural Engineering of the Civil Engineering Faculty will continue with an emphasis placed on the optimisation of the method of composite action in order to ensure the sufficient action between the concrete and the steel beam.

Acknowledgment

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