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REDISTRIBUTION OF FORCES BETWEEN REINFORCEMENT BARS AND PROFILES IN THE INTERMEDIATE SUPPORT ZONE IN THE DOUBLE SPAN, DOUBLE COMBINED BEAMS

REDYSTRYBUCJA SIŁ POMIĘDZY ZBROJENIEM PRĘTOWYM I KSZTAŁTOWYM W STREFIE PODPORY ŚRODKOWEJ W DWUPRZĘSŁOWYCH BELKACH PODWÓJNIE ZESPOLONYCH

A b s t r a c t

The aim of the study was to analyse the behaviour of the innovative precast beams, where the prestressed concrete and specially prepared steel profile were combined. This paper deals with the results of the tests and the analysis of redistribution of internal forces between reinforcement bars and steel profiles in the intermediate support zone in the proposed double span beams.

Keywords: composite structures, prestressed concrete, redistribution, steel profile

S t r e s z c z e n i e

Celem badań była analiza zachowania innowacyjnych belek prefabrykowanych, w których zespolono beton sprężony i specjalnie przygotowany profil stalowy. Artykuł przedstawia wyniki badań i analizę redystrybucji sił wewnętrznych pomiędzy prętami zbrojeniowymi, a profilem stalowym w strefie podpory środkowej w proponowanych dwuprzęsłowych belkach.

Słowa kluczowe: elementy zespolone, beton sprężony, redystrybucja, profil stalowy

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1. Introduction

The subjects of research were precast beams, where the prestressed concrete and specially prepared steel profile were combined. The presented specimen is an innovative solution, indirectly inspired by the concept of hybrid beams, produced by Preflex & Flexstress [1]. It was estimated that this kind of beams will be able to cooperate with monolithic slabs or precast slabs (e.g. hollow plates) with topping concrete. The study verified behaviour of elements in both situations. Additionally, it was predicted that these kind of beams could be applied in buildings with highly loaded binders or in road structures such as bridges etc.

2. Test specimens

2.1. Assumptions

The experimental tests comprised of six double span specimens. Each of the beams consisted of two single span precast, prestressed elements with a length of 4.0 m. Those precast beams were made by Ergon Poland company. The beams were loaded with two concentrated forces in every span (Fig. 1).

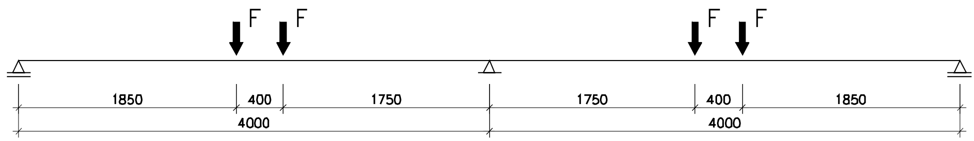

Fig. 1. Loading scheme

In the precast elements, a steel profile of a height of 290 mm height (Fig. 2) was used, composed of 1/2 H-section (top chord) and T-section (bottom chord). The top and bottom flange were connected with vertical double flat irons (6 × 25 mm), spaced equally at the beam length with 164 mm spacing. Double flat irons were welded on to the chords by fillet welds, with a thickness of 4mm and a length of 40 mm for the top chord and 44 mm for the bottom chord. The steel grade of the profile and the irons was 18G2 (steel grade due to Polish codes).

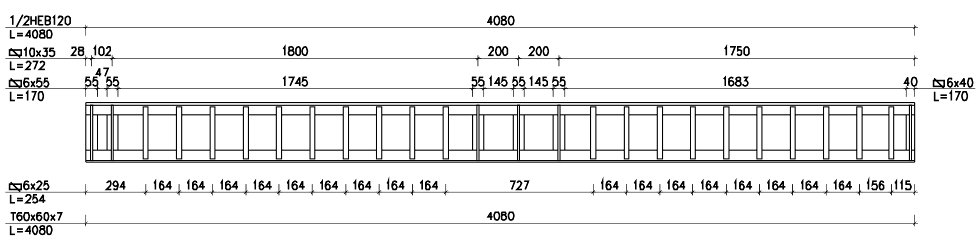

Fig. 2. The view of the steel profile used in tests

For the bottom longitudinal reinforcement, the prestressing strands were used. In Total, 8 strands Y1860 S7 with section A_p = 93 mm^2 were provided. Six of the strands were tensioned with a force of 138kN (σ_{pmo} = 1488 MPa) and the remaining two strands with a force of 20 kN (σ_{pmo} = 215 MPa).

As the upper reinforcement over the intermediate support the reinforcement bars were utilized. The steel had the characteristic yielding strength f_{yk} = 500 MPa. Three different upper reinforcement ratios were assumed. For the beams denoted as 301 and 311, the reinforcement ratio was 0.013 (4#20), for beams 302 and 312 the reinforcement ratio was 0.020 (4#20+2#22) and finally for beams 303 and 313 it was 0.026 (5#22+2#20). It should be noticed that this assumption imposes a different redistribution of internal forces for each of the beams.

Sections of the particular beams are shown on Fig. 3. For each beam, there are also presented the values of ultimate forces and calculated capacities with the assumption of full redistribution of internal forces.

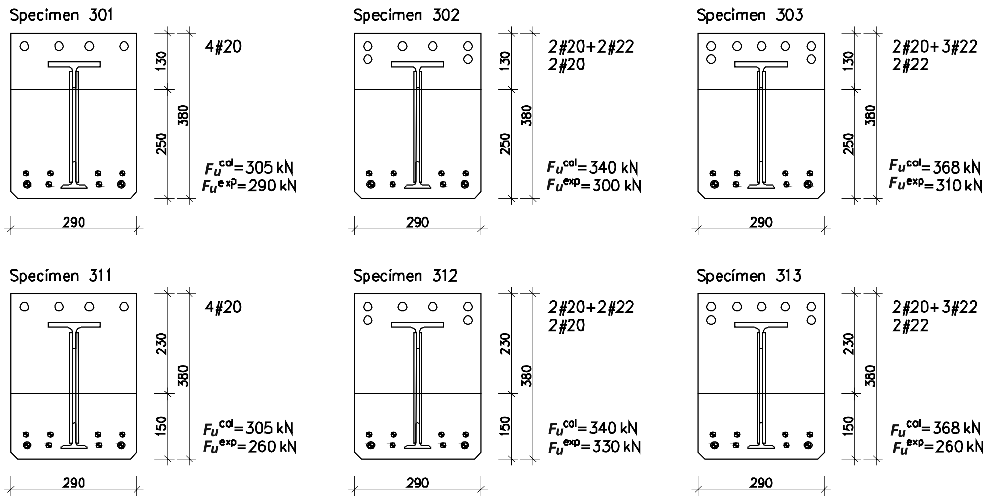

Fig. 3. Sections of the particular beam

As the reinforcement for shear there were used only exclusively flat irons with 6 × 25 mm section that connected welded to the flanges of steel chords. profile. During the designing of this reinforcement, it was assumed that the inclination of the compressed strut (according to the truss model) satisfy a condition cot θ ≈ 2 [2].

In the strands anchoring zone, the horizontal loops were applied, together with additional vertical stirrups made from 8mm bars to prevent damage due to prestressing. Those stirrups were also designed under point loads to protect concrete from local crushing. Along the entire length of the beam, the horizontal joints were used to connect precast concrete and steel profile.

In the supports zones, the additional bars with threaded ends were designed to prevent the separation of the precast concrete and the topping concrete. Figure 4 shows the location of those bars for the extreme and the intermediate supports. The delamination force was transferred by the double iron flats of the steel profile together with those bars.

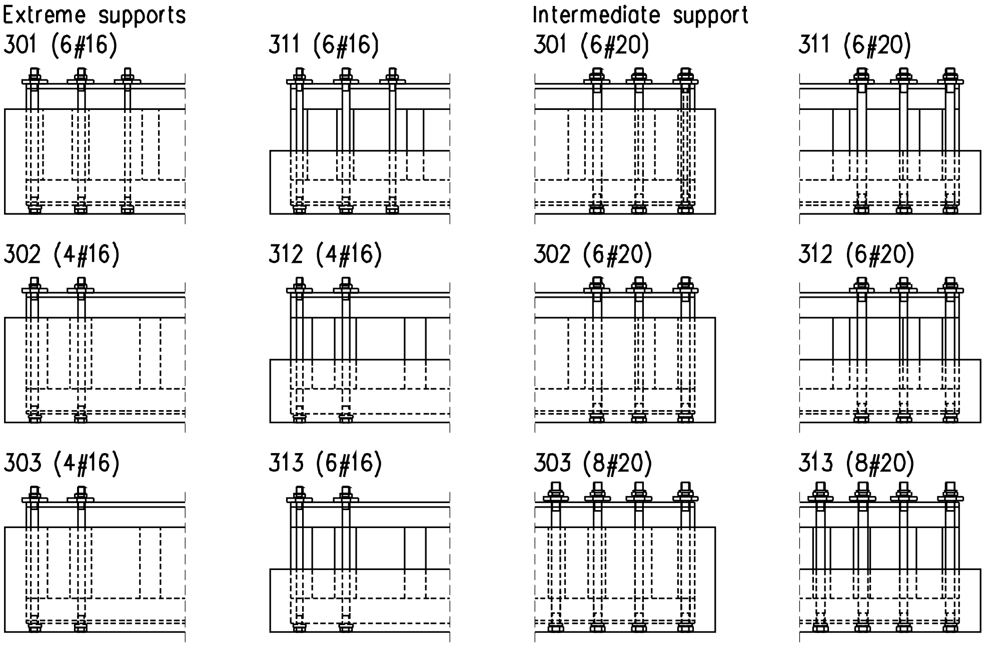

Fig. 4. Bars with threaded ends on the extreme and the intermediate supports

2.2. Materials

The elements were made in precast plant on the long production line with heat treatment. For all beams, the compressive strength of the precast concrete at the moment of the strands release was $f_{c,\mathrm{cube}}$ = 50.4 MPa, while during the tests the compressive strength was f_c = 74.4 MPa and $f_{c,\mathrm{cube}}$ = 84.5 MPa. The topping concrete at the moment of the study reached the compressive strength equal $f_c \approx 60$ MPa and $f_{c,\mathrm{cube}} \approx 70$ MPa.

The reinforcement steel had yielding strength equal f_y = 520 MPa, while the profile steel had f_y = 434 MPa for the 1/2 H-section and f_y = 289 MPa for the T-section.

Full description of the element's reinforcement, the characteristics of the materials and properties of the composite cross-section are presented in [3].

2.3. Test preparations

The precast elements were imported from plant to the Laboratory of the Department of Concrete Structures of Technical University of Lodz, where they were prepared for the study. Two identical elements were arranged in a line on four leveled bases. The gap between elements with a 10mm thickness was filled with the epoxy. In the bottom zone of connection between two beams, the additional irons were mounted due to transportation and assembly on the testing stand. After the application of the strain gauges on the steel profile, the upper reinforcement was added and the topping concrete was provided. Fig. 5 shows activities performed during the preparation.

Fig. 5. Photos of the preparations of elements in the research laboratory

After approximately two weeks, the system was boarded and erected on the test stand (Fig. 6). The beam was placed on three supports. The intermediate support was set as the non-movable and to make good contact between support and the bottom of the beam the cement mortar pad was made. The extreme supports were movable. One support was leveled to the intermediate support, the other one was free to rotate around the axis of the beam.

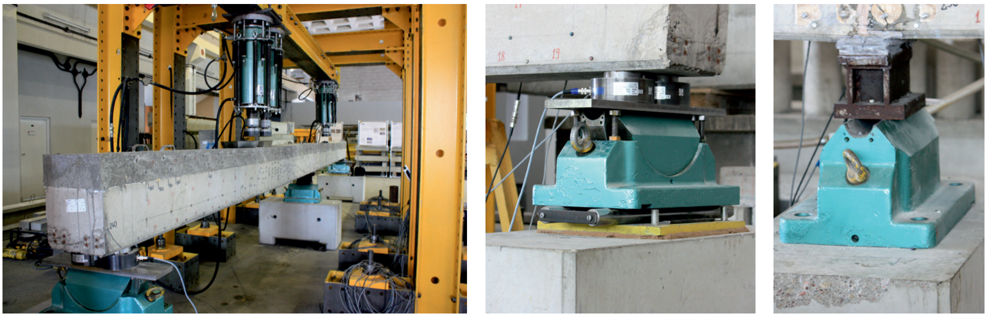

Fig. 6. Photos of the test stand, extreme and intermediate supports

2.4. Measurements

The mechanical extensometers with base 200 mm and 400 mm, electric strain gauges and Linear Variable Differential Transformers (LVDT) were used to measure the strains of the concrete and steel.

After the concreting of the precast specimen, just before the prestressing strands release (on the surface of the concrete at the bottom part of the steel profile and on the top chord of this profile) the special markers were applied to make the measurements bases. Reading measurements on these bases were performed before compression, immediately after compression and before performance the topping concrete. With these measurements, losses of prestressing force caused by elastic deformation of concrete and rheological losses in the period from compression to the tests were estimated.

Also after setting the final specimen on the test stand, just before studies, on the one side along the axis of the beam, the special markers were applied to measure the longitudinal strains – ε_x (Fig. 7) and the vertical strains – ε_y.

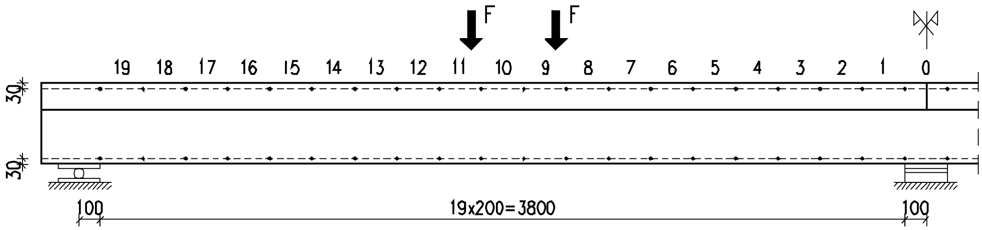

Fig. 7. Measurement bases – the mechanical extensometers with base 200 mm

In the tests, the load was applied by four hydraulic presses cylinders. The force was increased in fixed steps. After increasing and stabilizing of the force, the measurements were taken. The cracking development was observed together with crack width measurements. The total time of the one specimen test was about 5–6 hours.

3. Test results and analysis

In the intermediate *s*upport zone, calculations were carried out in the two sections – directly above the support for moment M_p ('0' base) and in the α – α section, which was 30 cm away from the support ('1' and '2' bases). Details of the connection of two precast beams and the reinforcement of the intermediate support zone are shown in Fig. 8.

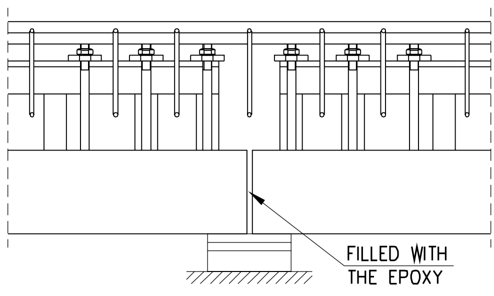

Fig. 8. Detail of the connection of two precast beams

Calculated strains were obtained by using the authors' program, which takes into account nonlinearity of concrete and reinforcement [4]. The calculations were performed twice. The first calculation took into account the presence of the reinforcement bars only in the intermediate support zone – relationship 'cal (fi)'. The second calculation also included the ½ H-section profile – relationship 'cal (fi + H)'. Fig. 9, 10, 11 present the comparison of the calculated relationships with real strains measured with mechanical extensometer (relationship 'exp').

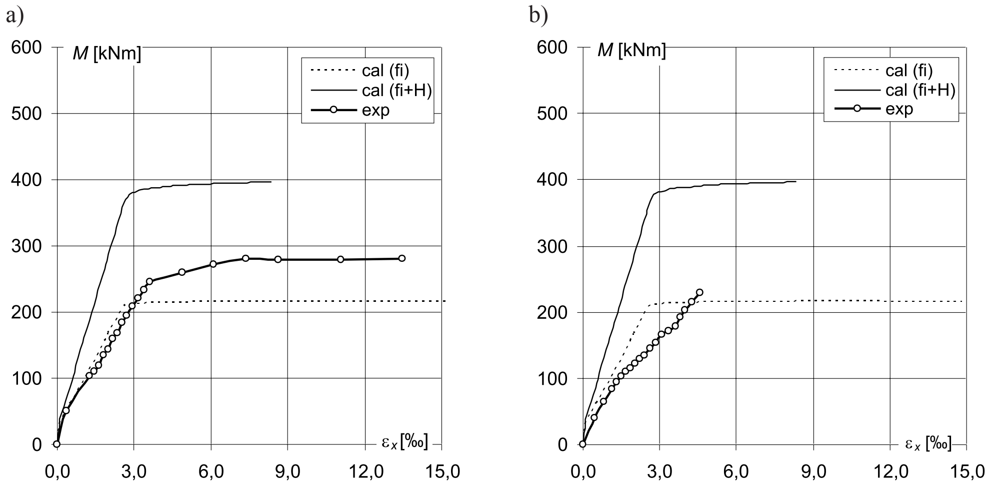

Fig. 9. Moment – strain relationships directly above the intermediate support a) beam 301, b) beam 311

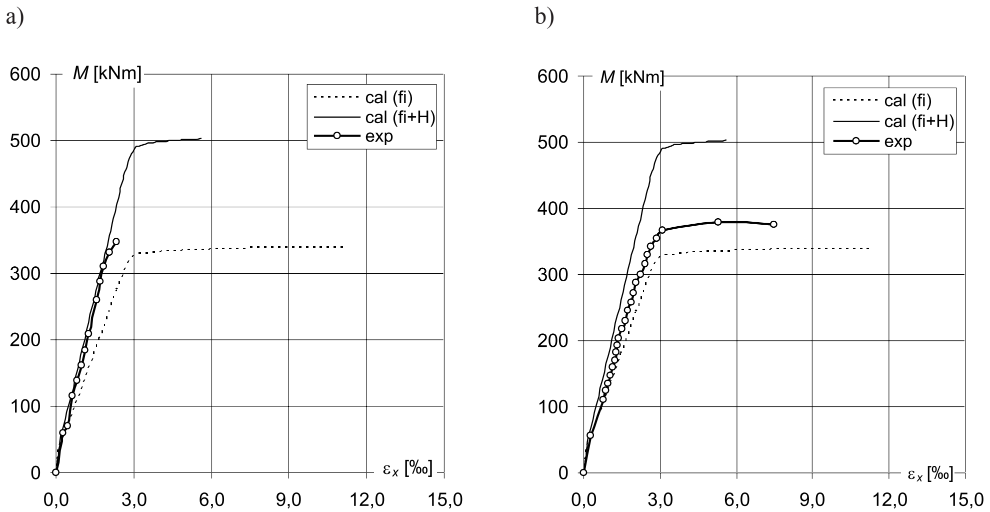

Fig. 10. Moment – strain relationships directly above the intermediate support a) beam 302, b) beam 312

The analysis shows clearly the influence of the steel profile at the strains directly above intermediate support zone, despite the fact that profiles were not continuous over the support. It is confirmed for different ratios of the upper reinforcement.

For example in beam 301, it is clear that after the beam reaches the capacity of the pure bar – reinforced section, local redistribution of the forces from bars to the steel profile emerges, and the capacity of the intermediate section is about 40% greater than the pure bar reinforced section. In specimen 303, this local redistribution of forces takes place before reaching the

a) b)

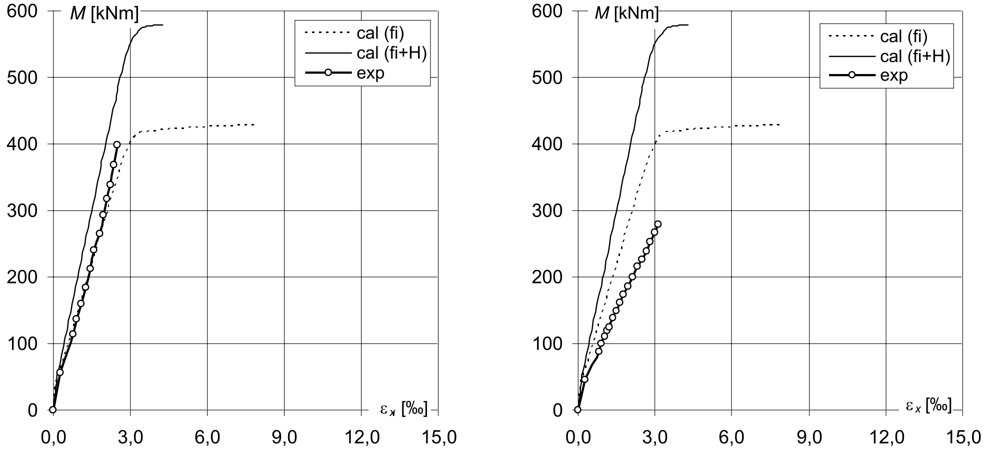

Fig. 11. Moment – strain relationships directly above the intermediate support a) beam 303,

a) b)

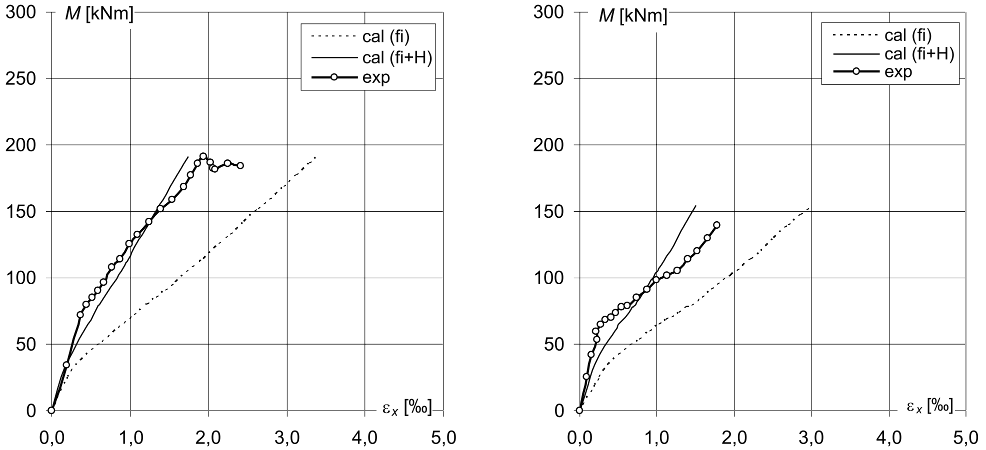

Fig. 12. Moment – strain relationships in the α-α section a) beam 301, b) beam 311

bending load capacity calculated for reinforcement bars only. It can be noticed by observing the relationship M – ε, which at first is concurrent with relationship for the reinforcement bars only and then after reaching the value 320 kN approaching to the analytical relationship taking into consideration also rigid steel profile. Even more clearly, the impact of rigid reinforcement on the support load capacity is visible in beam 302, where in the entire range of load, strains coincide with the calculations for bars and steel profile. Obviously, the impact of the used steel profile is observed in specimen 312. The support load capacity here was higher than the calculations for bars only by about 20%. The situation is different for beam 313. In this case, the capacity for the support cross-section is definitely lower even in comparison with the pure bar-reinforced section. This situation has come as a result of filling the gap between precast elements with too flexible a material which caused different behaviour of the compression zone. A similar situation was observed in beam 311 – although in this case, despite far less rigidity of the intermediate section observed after cracking, the load capacity is higher than when taking into account only the reinforcement bars. To sum up, it can be said that in each of the beams, to greater or lesser extent, the impact of a steel profile on the relationship $M - \varepsilon$ is visible. Of course, this also has an influence on the support load capacity.

There were also analyzed the relationships $M - \varepsilon$ for $\alpha - \alpha$ section, remote from the axis of the intermediate support of 30 cm. In Fig. 12–14, these strain diagrams are shown.

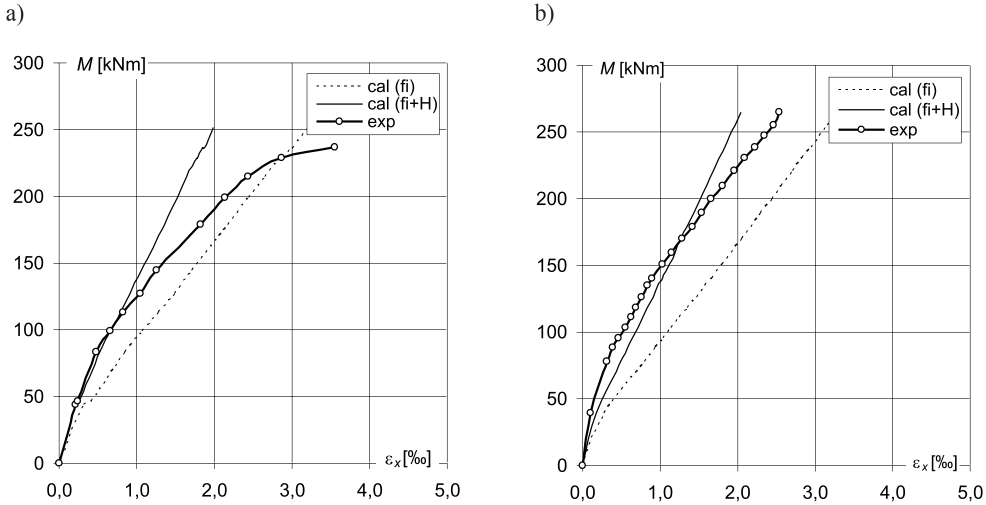

Fig. 13. Moment – strain relationships in the α – α section a) beam 302, b) beam 312

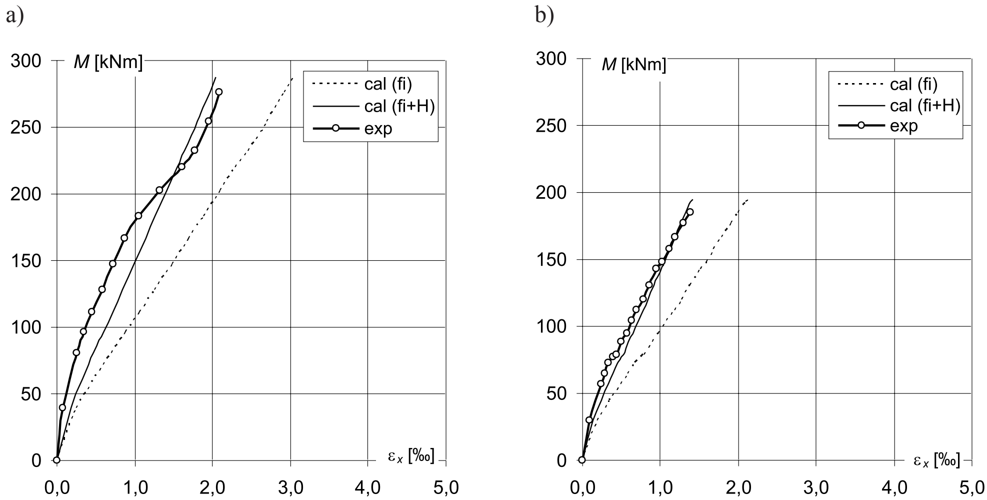

Fig. 14. Moment – strain relationships in the α – α section a) beam 303, b) beam 313

In the $\alpha-\alpha$ section, according to the assumptions of a truss model, the strains depend also on the value of the shear force and accepted safety ratio for shear, thus the accepted angle θ between the concrete compression strut and the beam axis perpendicular to the shear force, according to:

$$\cot\theta = \frac{1}{\eta} \tag{1}$$

where:

η – safety ratio for shear.

The diagrams show the results of tests on the background of calculated relationships. dependences. Calculations include the calculated angle determined on the basis of the cross-section area, the spacing between shear reinforcement, and the shear force assuming the full redistribution (see Table 1). In these cases, the calculations were also performed twice.

Table 1

The values of the shear forces and safety ratios for shear

No. of the beam	Shear force [kN]	Safety ratio for shear
301, 311	365	0,55
302, 312	435	0,46
303, 313	491	0,41

In beam 301, there was observed a clear conformity of measurements with calculated values taking into account the rigid reinforcement. In specimen 302, it was also evident up to about 40% of load capacity. However, beyond this level of effort, the actual strains were higher than it would be apparent from the calculations. It should be noted that just for this specimen in the section directly over the support measured strains were fully consistent with the calculations for the reinforcement bars and rigid steel profile. The actual stiffness of the element in the beam 303 in the $\alpha-\alpha$ section was higher, up to 80% of load capacity compared with calculations even for both reinforcements. Only when the effort was 0.8, the actual strains were similar to analytical ones. In the specimen 311 up to cracking, sectional rigidity was higher than that computed. Only after the crack strains were comparable with calculations for bars and rigid reinforcement. a similar relationship between the measurements and calculations can be observed in element 312. Instead, in the specimen 313 compliance between measured and analytical strains is directly perfect. To summarize, in the α-α section the influence of the rigid steel profile on the sectional stiffness is noticeable in each beam.

4. Conclusions

The conducted experimental studies have shown unequivocally that despite the lack of continuous rigid reinforcement directly over the intermediate support, the influence of the steel profile on both the load capacity and stiffness is readable in both examined sections (the support section and the α – α section). The authors believe that this is the effect of the use of the bars with threaded ends, which anchored the rigid profile (located in tension zone) in the compression zone.

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