EXPERIMENTAL STUDY ON EFFECTIVENESS OF INTERACTION BETWEEN PRE-TENSIONED HOLLOW-CORE SLABS AND CONCRETE TOPPING

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Abstract

Presently, hollow-core slabs are the most popular precast slabs used in many kinds of projects. Usually, medium-strength concrete is used as a cast in-situ topping, protected against shrinkage with fabrics of relatively thin wires. The long discussions at modifications of the Standards and finally very conservative recommendations in EN 1992-1-1:2004 caused the new situation. To treat such slabs as composite structures the transversal interface reinforcement or special shaping of precast members' surface is necessary. This is because live load is usually imposed not as uniformly distributed but as local (concentrated or linear). Therefore, the possible design shear resistance at the interface is very small, and transverse reinforcement must be introduced. Designers and contractors are against such rules, particularly in situations where topping is interacting with transversal and longitudinal parts surrounding precast elements. An experimental program was developed in order to clarify the actual behaviour of pre-tensioned hollow-core slabs with typical topping placed in-situ, without interface reinforcement or any special preparation of the interface. In the first step the elements over one year old were tested under instantaneous loading up to failure, while in the second step quite new elements – firstly subjected to long-term loading – were finally also tested up to failure. In both cases the interaction of topping was very good, up to the level of over 95% load at failure. The tests confirmed the observations from practice and opened the way to omit in particular cases the conservative rules given for all kinds of composite structures "with concrete cast at different times" (see p.6.2.5 in EN 1992-1-1:2004).

Streszczenie

Otworowe płyty strunobetonowe stanowią obecnie najpopularniejszy typ prefabrykowanych stropów, stosowanych w różnych typach konstrukcji. Zwykle płyty te są stosowane jako zespolone z betonem uzupełniającym średniej wytrzymałości, zabezpieczonym przeciwskurczowo siatką prętów o małej średnicy. Zgodnie z wymaganiami podanymi w EN 1992-1-1:2004, przekrój zespolony powinien mieć odpowiednie zbrojenie zespalające, lub powierzchnia styku powinna być odpowiednio ukształtowana. Zasady kształtowania przekroju zespolonego wynikają z założenia niekorzystnego oddziaływania obciążenia zmiennego, które może mieć charakter lokalny. Takie obciążenie może wywołać naprężenia rozwarstwiające przekraczające adhezję w styku. Ostrożne podejście normy budzi sprzeciw wytwórców prefabrykatów i projektantów, a argumentem z ich strony jest długa tradycja stosowania takiego rozwiązania. Przeprowadzone badania miały na celu ocenę zachowania się strunobetonowych płyt otworowych z udziałem betonu uzupełniającego, ale bez zbrojenia zszywającego i bez specjalnych zabiegów poprawiających przyczepność. Pierwszą serię badanych płyt obciążano doraźnie do zniszczenia. Były to płyty poddane bezpośrednim wpływom atmosferycznym przez ponad 1 rok na otwartym składowisku. Z kolei drugą serię płyt, pobranych bezpośrednio z produkcji, najpierw poddano obciążeniu długotrwałemu przez ponad 6 miesięcy, a następnie obciążono doraźnie do zniszczenia. W obu przypadkach współpraca płyt z betonem uzupełniającym była dobra. Aż do poziomu obciążenia stanowiącego 95% niszczącego nie nastąpiło rozwarstwienie w płaszczyźnie styku betonu uzupełniającego i prefabrykatu. Badania potwierdziły obserwacje płynące z praktyki i otwarły drogę do weryfikacji, w szczególnych przypadkach, ostrożnych wymagań Eurokodu 2 podanych dla wszystkich konstrukcji zespolonych „z betonem układanym w różnym czasie" (p.6.2.5 w EN 1992-1-1:2004).

Keywords: **Composite structures; Experimental analysis; Hollow-core slabs; Precast members; Pre-tensioned elements.**

1. INTRODUCTION

For the past decades the hollow-core slabs have become probably the most popular prestressed precast slabs used in many kinds of buildings. The range of applications is very wide – common use in office, school or commercial projects, multi-storey car-parks, various industrial buildings, as well as ordinary dwelling blocks is observed [1]. Sometimes these elements are used for footbridges and small secondary bridges. Such precast members are manufactured with total thickness ranged 200 mm to 600 mm, and used for spans of 6 m to 18 m.

According to specific manufacturing process (by concrete extrusion) since the beginning such members have been produced as pre-tensioned continuous long strips, without any longitudinal or transversal ordinary reinforcement. The strips are cut for required length of particular elements. Formally, such elements do not comply with recommendations in the currently applicable Standards.

For several decades pre-tensioned hollow-core slabs have been used in Europe and in North America with application of cast-in-site topping, usually 40 to 80 mm thick, without any special connectors between prefabricates and topping.

At present, prefabricates are most often produced with concrete C50/60, while supplementary concrete for in-situ topping is applied as C25/30 or C30/37 (according to the European Standard EN 206-1:2003 classes of concrete strength). As a rule the topping itself is protected with wire-mesh reinforcement, mainly against shrinkage.

The long discussions at modifications of the Standards and finally conservative recommendations given in EN 1992-1-1:2004 caused the new situation, particularly in the EU countries. To treat such slabs as composite structure the transversal interface reinforcement or special shaping of precast member top surface is necessary in many cases. This is because usually live load is imposed not as uniformly distributed but as local – concentrated or linear alike. Therefore, the acceptable design shear resistance at the interface is very small, and transverse reinforcement must be introduced. Very often designers and contractors are strongly against such rules, particularly in those situations when topping is interacting with transversal and longitudinal cast-in-site parts surrounding prefabricated parts. The detailed constructional recommendations could be neglected if such a decision is supported by representative tests.

Therefore, the experimental program was undertaken to clarify the real behaviour of pre-tensioned hollow-core slabs with typical topping placed in-situ, with neither interface reinforcement nor any special preparation of the interface.

2. EXPERIMENTAL PROGRAM

2.1. General aims

The full scale members, 7,0 m long, with depth of 0,32 m, were selected for tests. Such elements are used for floor slabs up to 12 m, according to the loading intensity, and up to 15 m for roofs.

The general aims of the tests were the following:

- Comparison of behaviour of slabs with and without topping, subjected to short-time (instantaneous) linear loading up to failure – for the elements with advanced process of losses of prestressing forces, i.e. one year old at least.
- Comparison of behaviour of slabs with and without topping, subjected to long-term loading – starting the tests for the elements relatively young, e.g. 1 to 2 months old.
- Comparison of behaviour of slabs with and without topping, after long-term tests (6 months), subjected to short-time linear loading up to failure.

The particular questions in the tests were related to two basic problems:

(a) Concrete mixture in precast members was used for the first time with the specific coarse aggregate (a kind of dolomite); it caused some changes in properties of hardened concrete in comparison with typical former mixtures used commonly in Poland (with quarzite, granite or basalt coarse aggregate).

(b) Adverse conditions were assumed for interaction between precast members and supplementary concrete. Neither side members fixing the wire-mesh reinforcement were used in topping, nor special processing of surface was done, apart from cleaning and wetting.

These two questions resulted from the long series of precast slabs planned for the large projects, to be applied in various situations.

2.2. Description of elements

The tested pre-tensioned hollow-core slabs with depth of 0,32 m, were prestressed with twelve seven-wire Y1860 strands with diameter of 12,5 mm. Four elements used in the tests were taken from different production series, with a bit different concrete mixtures. Concrete strength in slabs was controlled before tests by means of the Schmidt sclerometer. The data for concrete mixtures in the slabs and concrete strength is presented in Table 1.

Table 1.
Concrete strength and composition of concrete mixtures in the slabs [kg/m³]

Slab symbol	Age of slabs at final test [days]	Mean compressive strength f_{cm} [MPa]	Cement CEM I 52,5R	Sand	Dolomite		Water	SP	w/c ratio
				0÷2	2÷8	8÷12			
1	2	3	4	5	6	7	8	9	10
HCS-1	550	75,8	312,0	895,0	708,0	523,0	130,5	0,93 (*)	0,42
HCS-2	550	70,6	319,0	878,0	705,0	528,0	134,0	0,93 (**)	0,42
HCS-3	240	66,1	337,9	853,0	699,5	542,8	133,5	0,99 (**)	0,39
HCS-4	240	62,7							

SP – superplasticizers: (*) Chryso fluid CE40, (**) Rheobuild 2000PF

Two pairs of elements were considered. The first two slabs, HCS-1 and HCS-2, were stored on site for more than a year after casting, exposed to direct open air environmental conditions. Whereas, the next two slabs, HCS-3 and HCS-4, were stored in more-or-less constant laboratory conditions for less than 2 months after casting, and similar conditions were kept during all period of long-term tests.

For the test purpose, supplementary concrete C30/37 of thickness 50±2 mm was cast on one element out of each pair. The interface reinforcement was not applied; just basic surface cleaning and wetting was done before casting of topping concrete. Topping concrete was protected against shrinkage with the fabric (mesh 150×150 mm), made of Ø8 mm plain bars of ordinary steel (f_{yk} = 210 MPa). Topping concrete mixture composition and its properties are presented in Table 2.

2.3. Testing equipment

Two different loading modes were used for each group of elements. The elements HCS-1 (without topping) and HCS-2 (with 50 mm topping) were subjected to instantaneous linear loading up to failure, whereas the elements HCS-3 and HCS-4 were subjected to long-term loading. Finally, they were also tested up to failure, instantaneously. The elements were tested in simply supported scheme with the span length 6,9 m.

For the short-time tests, the linear loading was applied by four forces uniformly distributed along the slab width. Slab geometry and loading scheme is presented in Fig. 1.

For the long-term test, the loading was applied by sixteen concrete blocks, each weighting of about 11 kN. Therefore, the uniform load in long-term test was equal to 21,0 kN/m² (excluding self-weight). The long-term load equaled 35% of ultimate load and 58% of cracking load (in comparison to the test results for HCS-1).

The value of applied load force F and the related values, such as: slab deflection, concrete strains and slip of strands were measured. The inductive gauges, symmetrically attached along the slab sides, were used for measurement, according to scheme presented in Fig. 2.

Table 2.
Composition and properties of supplementary concrete (for slabs HCS-2 and HCS-4)

Mixture composition [kg/m³]				Consistency *VeBe(s)*	w/c ratio	Concrete properties		
						Strength		Modulus E_{cm} [GPa]
Cement CEM I 32,5	Sand	Gravel	Water			f_{cm} [MPa]	f_{ctm} [MPa]	
1	2	3	4	5	6	7	8	9
471	518	1209	196	8	0,42	45,0	3,1	34,9

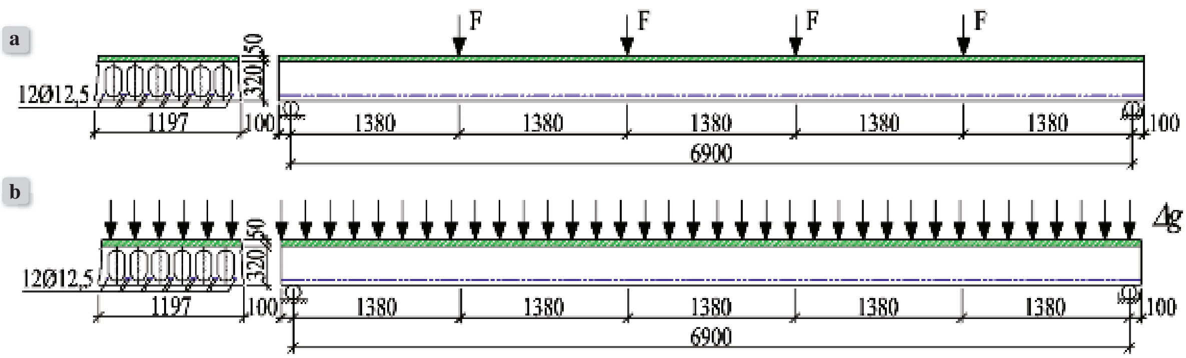

Figure 1.
Slab geometry and loading scheme: a) short-time loading, b) long-term loading

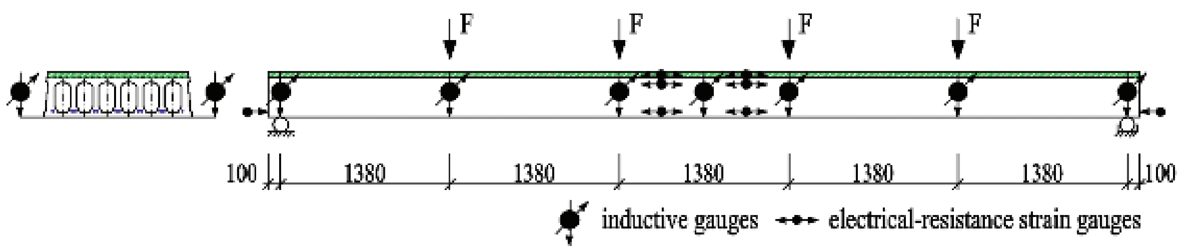

Figure 2.
Measuring system scheme

In short-time test, at each 12,5 kN increment of forces, the following data were recorded:

- deflection of slabs and slip of strands, by means of inductive gauges,
- concrete strains, by means of electrical-resistance strain gauges,
- location and width of cracks, particularly on the level of strands,
- the behaviour of the interface between the slab and supplementary concrete.

In long-term test deflection of slabs was measured for the period of 6 months.

3. RESULTS AND OBSERVATIONS

3.1. Short-time tests of elements under first loading

Before testing, the dimensions of elements were carefully measured. The reverse deflection of 5 mm characterized both elements HCS-1 and HCS-2.

Cracking of the HCS-1 slab (without topping) appeared approximately at the load level of $4F_{cr} \approx 250$ kN. It corresponds with a cracking moment $M_{cr} \approx 306$ kNm (excluding self-weight), whereas cracking of the HCS-2 slab (composite slab with topping) appeared at the load level of $4F_{cr} \approx 280$ kN ($M_{cr} \approx 345$ kNm).

The slab deflection at cracking was estimated for HCS-1 as 1/468 of span and for HCS-2 (with topping) as 1/488 of span. After cracking, deflection increase for the slab with topping was much lower in comparison to the other slab.

The slip of strands was not measured, so no de-bonding effect between strands and concrete occurred.

At failure, sudden shear at support zone was observed. Then, the strands were debonded and slipped away from concrete. The failure of slabs was not preceded by cracking at supports, but wide cracks were observed at midspan.

Failure of the HCS-1 slab (without topping) happened under the load of $4F_u \approx 440$ kN. It corresponds with a bending moment $M_u \approx 502$ kNm (excluding self-weight), whereas failure of the HCS-2 slab (with topping) happened under the load of $4F_u \approx 540$ kN ($M_u \approx 614$ kNm).

Until the failure, neither delamination nor any cracking were observed at the interface between precast slab (HCS-2) and its topping. It means that the shear resistance at the interface was quite sufficient. The delamination appeared just at failure, simultaneously to complete damage of the element.

3.2. Long-term tests

For the second group of slabs, subjected firstly to long-term load, reverse deflection of 9 mm (HCS-3) and 15 mm (HCS-4) was measured.

The load of about 21 kN/m² was applied to the slab and was kept constant for over six months (Fig. 3). The time-dependent effects were recorded during the whole test period, it means the results of concrete creep and shrinkage as well as strands relaxation. The temperature range was 15÷25°C and the average humidity was 65% (Fig. 4).

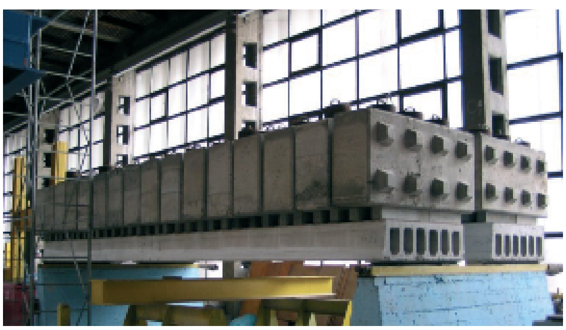

Figure 3.
HCS-3 and HCS-4 slabs subjected to long-term loading

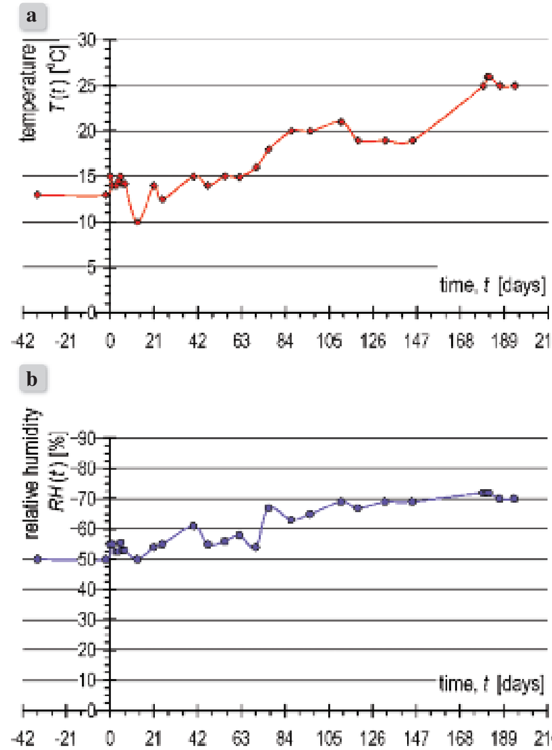

Figure 4.
Environmental conditions during the period of test: a) temperature, b) relative humidity

Main results of the test as of time-dependent deflections are presented in Fig. 5. Neither cracking of the slabs nor slip of strands was observed.

3.3. Short-time tests of elements after long-term loading

The slabs HCS-3 and HCS-4, first subjected to long-term loading (see 3.2), were subsequently tested under instantaneous loading up to failure. The summarized results for both groups of elements are presented in Table 3. There are compiled: load at first cracking, $4F_{cr}$ and ultimate load, $4F_u$; comparable concrete strain, $\varepsilon_{c,400}$; supplementary concrete strain, $\varepsilon_{cn,400}$; deflection of the slabs at midspan, a_{400}; and summarized width of cracks, Σw_{400}, at the chosen load of $4F$ = 400 kN.

The results obtained for the slabs HCS-3 and HCS-4 match well the results achieved for the slabs HCS-1 and HCS-2, which were subjected to first loading only.

The slip of strands was not measured, so, it was evident that no debonding between strands and concrete occured.

Some test results are presented below: comparison of bending moment – deflection relation (Fig. 6), development of summarized cracks width in the level of strands (Fig. 7), and development of strains on the top and on the bottom surface of the slabs (Fig. 8a,b).

Decrease in stiffness for the slabs first subjected to

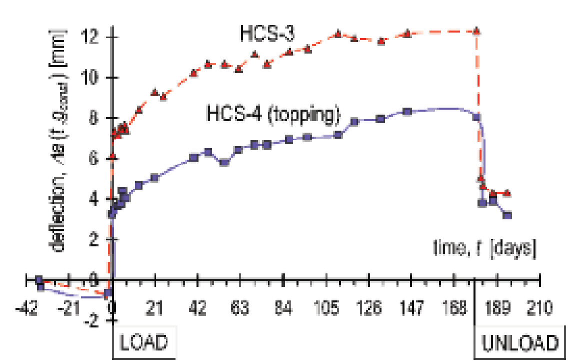

Figure 5.
Deflection development in time; average, at midspan

Table 3.
Results of short-time tests of elements (after long-term loading of HCS-3 and HCS-4)

Slab symbol	$4F_{cr}$ [kN]	$4F_u$ [kN]	$\varepsilon_{c,400}$ [‰]	$\varepsilon_{cn,400}$ [‰]	a_{400} [mm]	Σw_{400} [mm]
1	2	3	4	5	6	7
HCS-1	250	440	0,72	-	71,32	8,75
HCS-2	280	540	0,11	0,82	41,11	5,10
HCS-3	240	450	0,84	-	78,64	9,78
HCS-4	270	550	-0,04	0,66	50,40	7,25

long-term loading was observed. This happened due to higher losses of prestressing after 6 months of long-term loading equal to about 35% of ultimate load and 58% of cracking load expected for the slabs.

Failure mode for HCS-3 and HCS-4 (with topping concrete) slabs match very well previous observations. Sudden shear failure at supporting zone at the level loads almost equal to HCS-1 and HCS-2 (with topping concrete) slabs was observed.

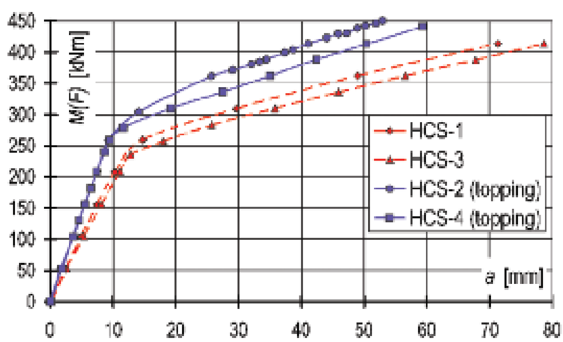

Figure 6.
Relationship deflection – bending moment under instantaneous loading

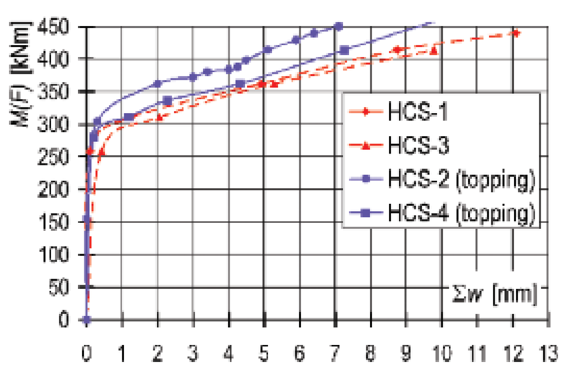

Figure 7.
Summarized cracks width, Σw, along the line of prestressing reinforcement

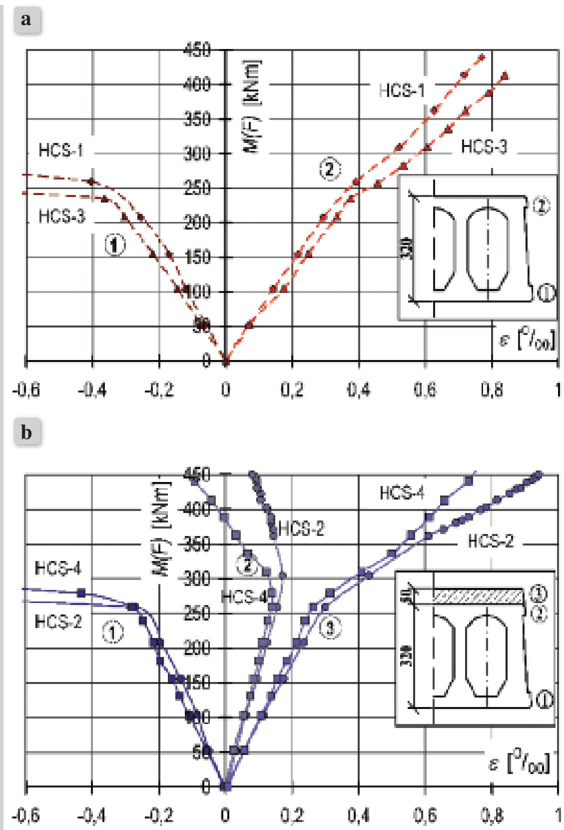

Figure 8.
Concrete strains at the top and the bottom of the slabs: a) HCS-1 and HCS-3; b) HCS-2 and HCS-4

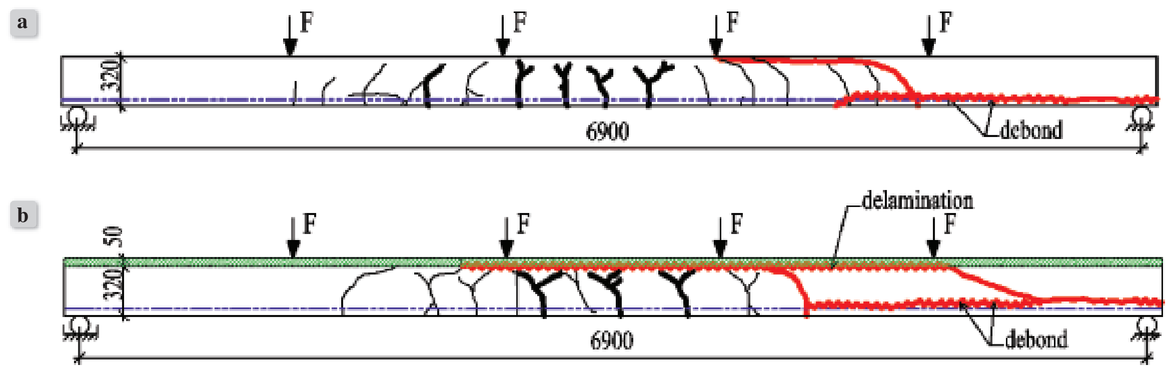

Figure 9.
The pattern of cracks at failure: a) HCS-1 (without topping), b) HCS-4 (with topping)

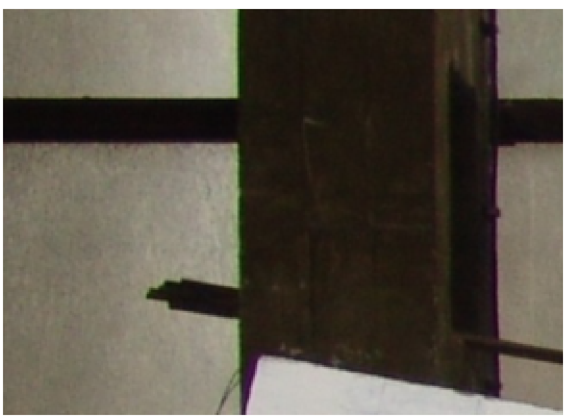

Figure 10.
End-zone of slab HCS-4 (with topping) at failure

The crack pattern at failure was very similar for all elements (Fig. 9). In all cases wide cracks at midspan appeared. For the slabs with supplementary concrete, delamination and debonding of strands occurred at supporting zone, as a final picture of a damage of the slab (Fig. 10).

In slabs with topping the high shear resistance at the interface was confirmed. Neither cracks nor delamination effects at interface were observed, up to the final phase of failure.

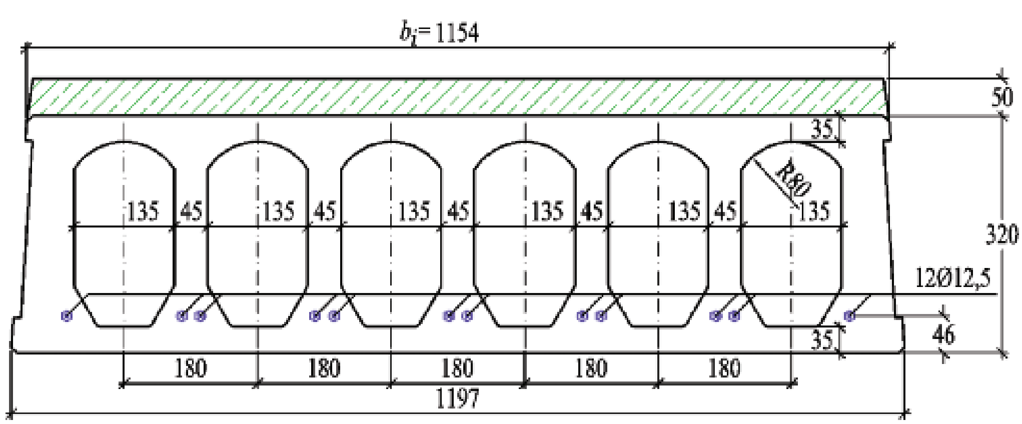

Figure 11.
Nominal cross-section of slab with topping and location of strands

4. DISCUSSION OF THEORETICAL SHEAR RESISTANCE AT INTERFACE

When calculating the cross-sectional characteristics of the slabs, the nominal section was used (Fig. 11). In this way, the cross-sectional area A_{cs} = 196410 mm^2, first moment of area S_{cs} = 3,186×10^7 mm^3 and second moment of area I_{cs} = 2,846×10^9 mm^4 were obtained.

In EN 1992-1-1:2004 the design value of the shear stress at the interface is calculated from the Eq. (4.1):

$$v_{Edi} = \frac{\beta \cdot V_{Ed}}{z \cdot b_i} \tag{4.1}$$

where: β is the ratio of the longitudinal force in the new concrete area and the total longitudinal force in the compression zone, V_{Ed} is the transverse shear force, z is the lever arm of composite section, b_i is the width of the interface (b_i = 1,154 m).

When analyzing the presented test results under instantaneous load, mean compressive strength f_{cm}= 70,6 MPa (see Table 1) was used as a representative value of concrete compressive strength. Considering the transverse shear force at the failure at support zone in the test V_{Eu} = 2F_u = 270 kN, Eq. (4.1) gives the value of the shear stress at the interface v_{Eui} ≅ 715 kPa.

The shear stress obtained from Eq. (4.1) is compared in EN 1992-1-1:2004 with the design shear resistance v_{Rdi}. The design shear resistance at the interface is obtained from Eq. (4.2):

$$v_{Rdi} = cf_{ctd} + \mu\sigma_N + \rho_j f_{yd}(\mu \sin\alpha + \cos\alpha) \le 0{,}5\nu f_{cd} \tag{4.2}$$

Factors c and μ depend on the roughness of the interface. The surface of precast members was classified as smooth; for free surface left without further treatment after vibration: c = 0,35 and μ = 0,6. As more adequate to the test results, the characteristic tensile strength f_{ctk} = 2,0 MPa (for C30/37 topping – the concrete with the lower strength) was used in Eq. (4.2).

For unfavourable loading with linearly concentrated forces the shear resistance at the interface is limited to concrete friction only. Normal stress σ_N was in this case represented only by the self-weight of topping (other forces were linearly concentrated), therefore its value is practically negligible, σ_N ≅ 0 MPa. Furthermore, there was no reinforcement crossing the interface in tested slabs (the transverse reinforcement ratio ρ_j = 0).

Characteristic shear resistance v_{Rki} = 700 kPa, obtained from Eq. (4.2), according to the procedure recommended in EN 1992-1-1:2004, is almost equal to 715 kPa – the stress obtained at ultimate load in the laboratory test. Whereas, during the test until slab failure neither cracking nor delamination were observed at the interface, the test results proved that the formula for calculating shear resistance seems to be inadequate in real work conditions for such a kind of composite hollow-core slabs, so, it gives considerably high safety margin.

Test results showed that in case of much developed interface surface (which is typical for slab floors) shear resistance at interface is much higher in comparison to idealized shear resistance estimated according to the EN 1992-1-1:2004. This proves that rules given in EN 1992-1-1:2004 may be omitted in particular situations.

5. RECAPITULATIONS AND CONCLUSIONS

Main results of the tests of prestressed hollow-core slabs are presented. The elements were tested in simply supported scheme, which is a typical static scheme for these elements in real structures. The research was focused on slabs behaviour and particularly on the advantage of composite section of the slab with 50 mm thick topping concrete. Adhesion and friction at the interface were examined.

Concrete topping was reinforced only with the use of minimum shrinkage reinforcement for crack control. No additional transverse reinforcement was provided, just basic slab surface preparation was done by means of its cleaning and wetting.

As far as prestressed slabs are considered, losses in prestressing must not be neglected. Therefore two groups of elements were subjected to tests, relatively young slabs, 1 to 2 months old, and the elements with advanced process of losses, tested after more than a year from casting. For complete clarifying of the slabs' behaviour the elements were subjected both to short-time loading up to failure and to long-term loading for 6 months.

The elements during tests behaved almost exactly as it was expected. Increase in deflection was observed, especially noticeable for the first 2 months of load

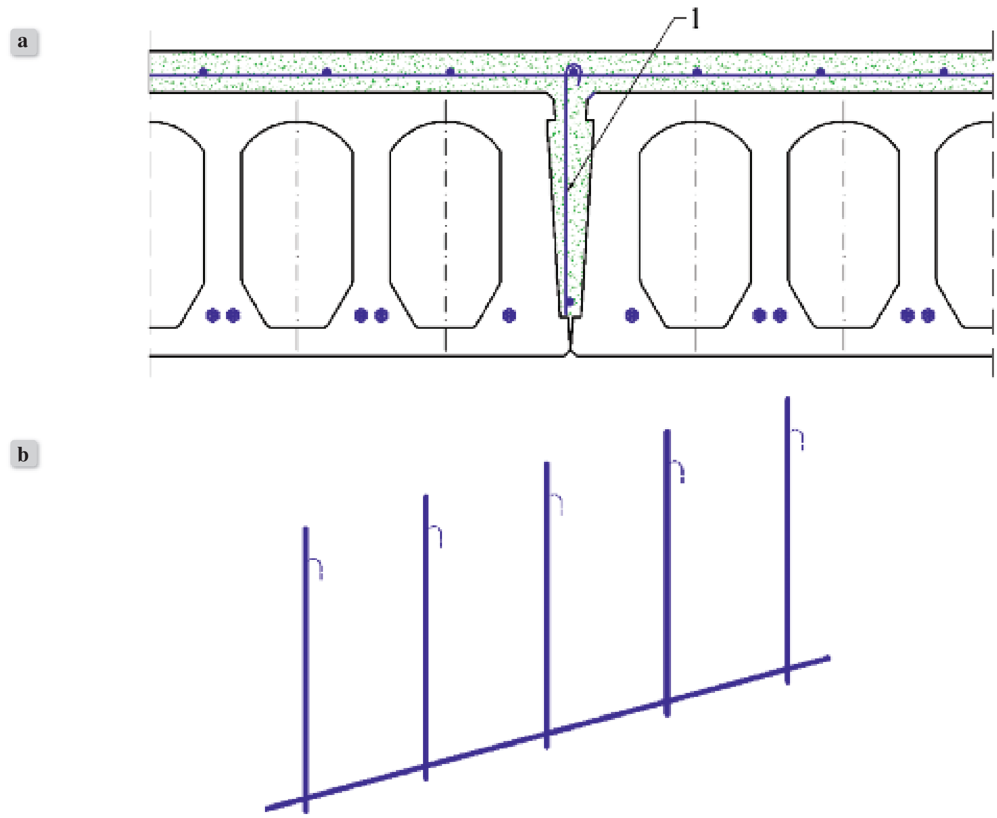

Figure 12.
Additional simple protection: ties for horizontal shear placed in longitudinal gaps:
a) cross-section; 1-transverse reinforcement, b) example of prefabricated transverse reinforcement (1)

application, as a result of time-dependent phenomena. The 6 months loading influenced behaviour of the slabs during final short-time test. Due to higher losses in prestressing, stiffness of the slabs slightly decreased.

During the short-time test under instantaneous loading no early damage was observed and debonding effects of the strands either. Supplementary concrete 50 mm thick became evidently advantageous. Even without any additional surface processing and with omission of tie reinforcement at interface neither cracking nor delamination were observed between the slab and topping concrete up to the level of over 95% of failure load. Worth mentioning is the fact that satisfactory behaviour of composite section was achieved under unfavourable load of linearly concentrated forces.

Cracking loads and ultimate bending moments for tested slabs were relatively high.

The following were measured for the slabs with topping concrete compared to plain hollow-core members:

- ultimate bending moment increase of more than 22%,
- cracking moment (the first cracks width of 0,05 mm) increase of almost 13%.

In general, the tests proved high load capacity, cracking resistance and stiffness at bending of prestressed hollow-core slabs. Further improvement in general structural performance was observed for composite element with topping concrete. Not long ago some other doubts were clarified for hollow-core slabs [2], [3]. All these research works made more precise and safer application of such elements possible nowadays.

Despite evident good interaction between slabs and topping observed almost up to the failure, in real structures additional protection – if possible – should be provided. This is particularly recommended in those cases when the conditions during construction works are uncertain, or when design assumptions concerning characteristics of loading are not clearly determined. Fig. 12 shows a simple method of additional reinforcement protecting the interaction between hollow-core slabs and thin topping.

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