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Research paper

Analysis of the limit states of a rib-and-block floor based on beams with steel truss

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Abstract: From among the many technological solutions for floors in residential buildings and in small public buildings very often rib-and-block floor systems are selected. The main argument convincing project owners to choose such solutions are their clear advantages, mainly the uncomplicated construction technology, no need to use advanced tools and equipment and the low cost. The introduction of harmonized European guidelines in the form of the EN 15037 series of standards made it necessary to introduce changes in the designing and constructing of these commonly used floor systems, whereby their manufacturers had to update their offers. This paper discusses the requirements and the main changes introduced by the EN 15037 series of standards in comparison with the previous guidelines and general calculations of reinforced concrete structures according to Eurocodes. Analyses concerning bending and shear ultimate limit states and serviceability limit states are presented. Critical points in the design of rib-and-block floors are indicated and the results of exemplary calculations for a selected floor are presented. In addition, a comparative analysis concerning the difference in the consumption of reinforcing steel for a selected floor system designed according to respectively the current and previous guidelines is carried out and its results are discussed.

Keywords: active deflection, limit state, reinforced concrete, rib-and-block floor

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1. Introduction

Rib-and-block floor systems are one of the basic technical solutions used for horizontal partitions in buildings. This floor technology is most often used in residential buildings, but also in some public buildings. In practice, rib-and-block floors are the most popular ribbed floor systems, mainly owing to the technological factors: small dimensions, light weight of the floor components, no need to use complicated structural floor reinforcement and limited or eliminated use of additional floor formwork.

As regards their structure, the most popular rib-and-block floors consist of precast beams performing the load-bearing function (as they contain reinforcement) and prefabricated ceiling blocks or other inserts filling the gaps between the beams and ensuring a constant spacing of the latter and a proper stiffness of the floor during its construction. The final basic component of rib-and-block floor is concrete which fills voids and integrates all the floor system components, whereby a reinforced concrete rib between the blocks and a thin concrete slab over the latter are formed. The precast beams are made as, i.a., reinforced concrete beams with steel truss, ceramic beams with steel truss or as prestressed concrete beams. There are also floor designs in which the reinforced concrete beams are fully prefabricated. Their variants include designs with protruding connecting rebars (enabling rib-slab interaction) and without such a reinforcement (no rebar-slab interaction). Rib-and-block floors are currently the most popular group among ribbed floor systems and are manufactured in many types differing in their load-bearing capacity, rigidity, weight, ceiling block material, sound reduction, thermal performance and in many other parameters. Exemplary commonly used rib-and-block floor systems are: Fert (ceramic ceiling blocks on ceramic-concrete beams with steel truss), Teriva (lightweight concrete ceiling blocks on reinforced concrete beams with steel truss) and Rector (lightweight concrete ceiling blocks on prestressed beams). Cross sections through the above-mentioned typical rib-and-block floors area shown in Fig. 1 (Fert floor – a, Teriva floor – b, Rector floor – c).

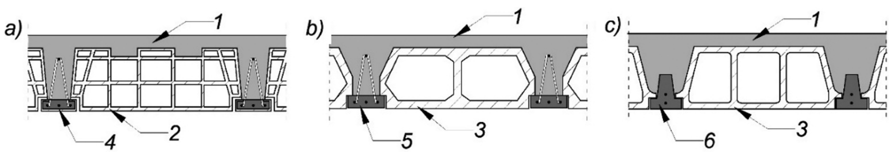

Fig. 1. Examples of rib-and-block floors (1 – concrete overlay, 2 – ceramic ceiling block, 3 – lightweight concrete celling block, 4 – prefabricated ceramic-concrete beam with steel truss, 5 – prefabricated concrete beam with steel truss, 6 – prefabricated prestressed beam)

Rib-and-block floor systems are not a popular research topic. Shear and debonding are most often discussed in the case of such floors [3, 4]. Computational analyses, practical experience and investigations of debonding in rib-and-block floors show that debonding between the beam concrete and the concrete overlay is not critical for the floor's load-bearing capacity despite the fact that the area of this contact is small. The failure mechanism here is more complex, involving also concrete zones beyond the beam concrete/concrete overlay contact near the support zone [3]. Studies also show that the popular models of shear in rib-and-block floors do not take into account the interaction of the slab formed in the concrete overlay with the ribs, whereby the floor's shear strength is underestimated. When the concrete overlay slab is taken into account, the rib failure mechanism changes as shear failure does not occur and the main structural floor components undergo flexural failure [4]. Theoretical analyses of the width of cracks in the prestressed beams of rib-and-block floor systems indicate that the calculation methods typically applied to prestressed reinforced concrete structures do not always yield correct results. In the case of the complex cross sections which occur in prestressed floor beams, complicated calculation modifications need to be introduced. The modifications have been simplified by means of a design model and a crack width checking method based on Tables and diagrams [5]. Also attempts at creating numerical models covering the floor rib-slab interaction, indicating possible material savings when appropriate interactions between the floor components are taken into account, are reported [6]. Rib-and-block floor systems are also described with regard to their sound reduction [7] and dynamic response [8]. Considering the typicality of rib-and-block floor designs and the calculation complications involved in determining the load-bearing capacity of such floors, attempts are made to tabulate the calculation results in order to facilitate the design of rib-and-block floors meeting relevant local standards [9].

2. Rib-and-block floors in light of regulations

Rib-and-block floor systems are manufactured mainly by local manufacturers who very often are slow in making their products conform to the changing regulations and guidelines. This is also due to the general attitude of construction process participants to the conformity of structural components with the standards in force. Surveys carried out in Poland [1] show that only 9% of the respondents think that the conformity of a floor system to the latest standards is an important criterion for their choice. According to the latest standard guidelines rib-and-block floor systems should be calculated consistently with and adjusted to the harmonized system of Eurocodes. There are descriptions [10] in which rib-and-block floors were treated as a conventional rib-slab reinforced concrete structure and so all the calculations were done in accordance with Eurocode 2 [11]. However, one should note that rib-and-block floors are not described in detail in the Eurocodes and so separate standards: EN 15037-1 [12] and EN 15037-2 [13] where the requirements which such floors must meet, from the point of view of both the floor system manufacturer (general product requirements and requirements concerning product properties and geometry) and the floor designer (calculation methods, design formulas and requirements), are precisely defined.

Many floor system manufacturers in Poland continue to provide systems noncompliant with Eurocode 2 [11] and with harmonized standards EN 15037-1/2 [12,13]. The standards describing ways of calculating and constructing rib-and-block floors [12, 13], oblige manufacturers of prefabricated floor units to comply not only with the design requirements, but also with several requirements concerning the floor geometry. Standard EN 15037-1 [12] also stipulates, i.e., the minimum thickness of the concrete cover on the block's top surface (40 mm at the beam spacing of less than 700 mm), the minimal strength class of the concrete of which the precast beams are made (C25/30) and the minimal strength class of the overlay concrete placed in situ (C20/25). Furthermore, requirements for resting the beams on the supports and the layout of the concrete overlay reinforcement (i.a. the necessity to reinforce the concrete overlay with mesh on the whole floor area or to use synthetic fibre reinforcement) and additional requirements concerning the floor serviceability limit state (limitation of active deflections) have been imposed on floor designers. Standards [11–13], introduce many restrictions on the production, design and construction of rib-and-block floors, whereby the manufacturers of floor systems are forced to make changes in the offered range of prefabricated units and in the load capacity Tables for the particular floors.

3. Floors covered by this case study

An analysis of rib-and-block floors compliant with all the design requirements stipulated in standards [12, 13] was carried out. A typical case of floors with cinder blocks resting on precast reinforced concrete beams with steel truss (floor system trade name Teriva) was studied. Depending on the assumed floor geometry a code name in the format "floor thickness/beam spacing" (e.g. a 240/600 floor – floor thickness = 240 mm, beam spacing = 600 mm), identifying a particular case of floor, was adopted. The analysed cases are presented in Fig. 2.

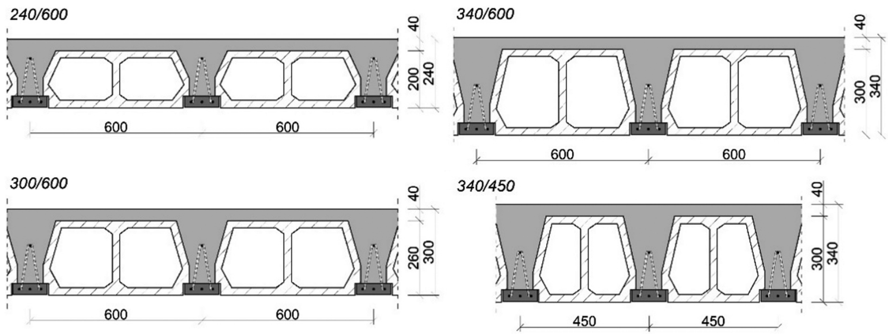

Fig. 2. Sketches of cross sections of analysed floors

All the calculations were described and carried out in accordance with the guidelines of current standards [11–14]. When determining the reinforcement required for the beams, reinforcement standardization for each span of the beam was assumed (which is a common practice among floor system manufacturers) to avoid an error (a given type of beam of a given span can be used in each offered type of floor). The calculation results were compared with the previously offered floors with similar specifications, but noncompliant with the valid (valid before the introduction of Eurocodes) guidelines. The analyses of the rib-and-block floors were carried out with regard to the ultimate limit states for bending and shearing (in the full range specified in standards [11, 12]) and to the serviceability limit state for deflection.

4. Bending

The reinforcement of all the beams consisted of a welded 3D steel truss and, if need be, an additional rebar situated between the bottom truss rebars. For technological reasons three types of trusses, as shown in Table 1, were adopted. When selecting a particular truss and, if need be, using the additional rebar, efforts were made to ensure economical, but safe and rational, utilization of the reinforcement in each of the beams. The diameters of the bottom truss rebars were gradated at every 4 mm to minimize a possible error during manufacture. It should be noted that according to current regulations [11, 12] no more than one additional rebar can be used between the bottom rebars of the truss. Previously, often two rebars would be used. This should be regarded as a structural error which can lead to the development of longitudinal cracks in the beams (especially at the stages of manufacture, transport and assembly) due to the small distances between the bars.

Table 1. Welded trusses used

	Type 1	**Type 2**	**Type 3**
Top/diagonal/bottom rebars	2 # 6 / 1 # 5/ 1 # 8	2 # 10 / 1 # 5 / 1 # 8	2 # 14 / 1 # 5 / 1 # 8

The following assumptions (conforming to [11]) were made in the ultimate limit state analyses:

– plane sections remain plane after deformation,
– the strain in bonded reinforcement, whether in tension or in compression, is the same as that in the surrounding concrete,
– the tensile strength of the concrete is ignored,
– the design cross-section with the most essential dimensions is shown in Fig. 3,
– the stress-strain relationship assumed for the reinforcing steel, according to [11],
– strains (as shown in Fig. 4) were assumed in the cross-section at a rectangular distribution of stress in the compressed concrete.

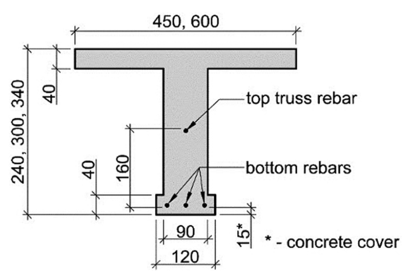

Fig. 3. Design cross-section (dimensions in mm)

Exemplary strain distributions for the floor height of 340 mm and the beam spacing of 450 mm are shown in Fig. 5. These distributions take into account the two extreme reinforcement ratios used for this floor (the minimal reinforcement in the form of two 6 mm rebars and the

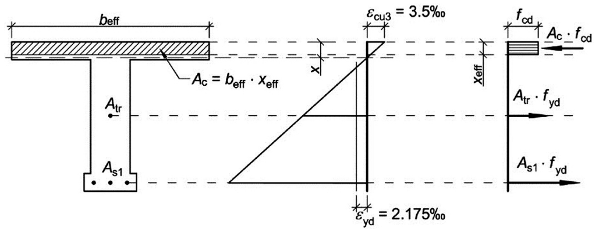

Fig. 4. Strain and stress distribution in cross-section

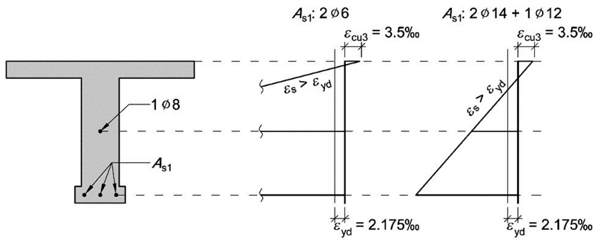

Fig. 5. Exemplary strain distributions for floor 340/450

maximal reinforcement in the form of two 14 mm rebars and an additional 12 mm rebar). The greatest height of the compression zone, and so the lowest strains in the reinforcement, were obtained in the case of the maximum reinforcement. One can notice that even then the tensile strain in the top truss rebar exceeds plastic strain $\varepsilon_{\mathrm{yd}}$. Therefore when checking the bending capacity/designing the reinforcement one can assume that the tensile stress in the top truss rebar amounts to f_{yd} (thus this rebar is fully utilized). The share of this rebar in the bending capacity, depending on the type of floor, is shown in Table 2. The shares are the larger, the smaller the reinforcement area at the beam's bottom edge and in the extreme case they reach up to around 30%. It should be noted that when designing such structures, customarily only the reinforcement concentrated at the bottom edge is taken into account. Considering the results of the analyses, this can be regarded as an uneconomical practice (especially that we are dealing here with the use of floors on a massive scale). It is essential, however, each time to check the distribution of strains in the cross section to determine whether the strain in the top truss rebar is not lower than plastic strain ε_{yd}.

Table 2. Share of top truss rebar in floor bending capacity

Type of floor	**240/600**	**300/600**	**340/600**	**340/450**
Share of rebar in bending capacity [%]	3–21	4–28	5–31	5–31

The design of the new typeseries of beams in accordance with European standards [11, 12] and with the above assumptions, in the case of some beams brought savings in reinforcement in comparison with the previous typeseries. It should be also noted that the new floor can carry loads by 0.5 kN/m^2 greater than the load carried by the old floor. The differences between the total mass of respectively the new reinforcement and the old reinforcement for the particular beams are graphically presented in bar chart (Fig. 6). Positive values represent a reduction in reinforcement mass (savings, despite the loads increased by 0.5 kN/m^2) while negative values represent an increase in reinforcement mass.

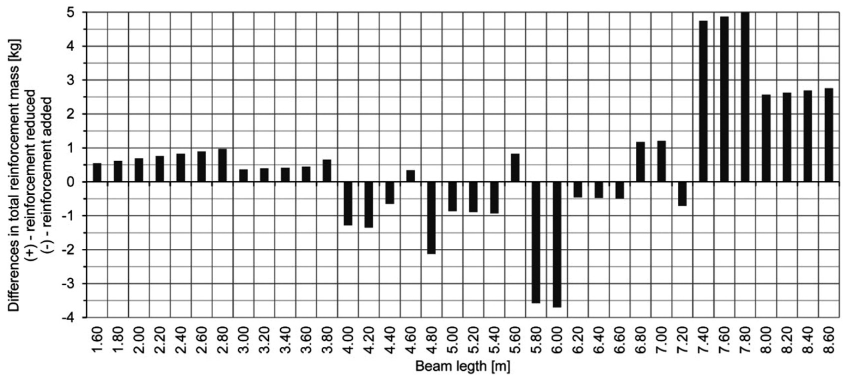

Fig. 6. Differences in total reinforcement massxx

One can see that in the case of beams up to 3.80 m long, the current reinforcement is lighter than the previous one by about 0.5–1.0 kg. In the case of beams with a span of 4.00–7.20 m, there is a need to use more reinforcement (maximally 3.70 kg more for the beam with the span of 6.00 m), which is dictated mainly by the greater loads. Unfortunately, these beam spans are most commonly used. In the new version the amount of reinforcement in the beams with the greatest lengths (above 7.20 m) is reduced by 2.5–5.0 kg.

It should be noted that standard [12] gives the simplified dependence for floor beam bending capacity with global safety factor for the ultimate moment ($\gamma_R = 1.10$).

Standard [12] proposes to use a global safety factor of 1.10 and do the calculations using the characteristic yield point of the reinforcing steel. Whereas the approach proposed in this paper conforms to standard [11] and so does not take the global safety factor into account. Instead, the design yield point of the steel (reduced by a factor of 1.15) is assumed. Hence the approach proposed in this paper can be regarded as more conservative. But its advantage is that it takes into account the top truss rebar. Standard [12] takes into account only the reinforcement concentrated at the beam's bottom edge. Ultimately, using the presented approach, lower values of the required reinforcement are obtained.

5. Serviceability limit state

The same serviceability limit states as for reinforced concrete structures designed according to [11] should be checked for rib-and-block floors designed according to [12]. In the case of both stress limitation and cracking limit states, standard [12] refers to the procedure given in [11], but it gives a different procedure for checking allowable deflections.

In the case of rib-and-block floors, according to [12] one should limit active floor deflections in order to avoid damage to, e.g., partition walls. Active deflections constitute a difference between the total deflection and the deflection after applying a brittle finish to the floor. The standard specifies three successive values of allowable active deflections: L/500 (L – the floor span in the clear between the supports, but it is not the effective span as in [11]) for masonry partition walls and/or a brittle floor finish, L/350 for other partition walls and/or a nonbrittle floor finish and L/250 for roof elements. It should be noted that an active deflection cannot be reduced by a pre-camber since in this case it is not the ultimate displacement value which is important, but the fact of the roof deflection itself. Active floor deflection is the condition which is usually most difficult to meet when designing rib-and-block floors.

A procedure for calculating the active deflections of a rib-and-block floor is presented in Annex E to standard [12]. The loads taken into account in the calculations are: the dead weight of the floor, the long-lasting loads for which active deflection checking is done (partition walls, suspended ceilings, etc.), the long-lasting loads applied to the floor before and after a brittle finish is applied to the latter and the long-lasting and short-lasting part of the live loads. The active floor deflection is calculated from the general formula:

$$f_a = w_t - w_a \tag{5.1}$$

where: f_a – the value of the active floor deflection, w_t – the value of the total deflection, w_a – the value of the deflection after applying a brittle finish to the floor.

Total deflection w_t is calculated taking into account all the mechanical loads and the shrinkage of the concrete. Deflection w_a is generated by the dead weight of the floor, the long-lasting load originating from the elements resting on the floor (e.g. partition walls), the long-lasting load applied before applying a brittle finish to the floor and the shrinkage of the concrete. Moreover, when calculating deflection w_a, also time t between unpropping and applying a brittle finish to the floor is taken into account. Deflection w_a is calculated from the formula:

$$w_a = w_1 + \psi(w_2 - w_1) \tag{5.2}$$

where: w_1 – the value of the deflection after applying a brittle finish to the floor, immediately after unpropping, w_2 – the value of the deflection after applying a brittle finish to the floor, a very long time after unpropping, ψ – an interpolation coefficient whose value is within the interval of 0–0.5 (calculated on the basis of time t).

Exact relations for deflections w_t, w_1, w_2 are given in [12]. It should be mentioned that in the formula for deflection w_2 in [12] (the Polish version) there is an error – the summand representing shrinkage should not be multiplied by 2/5.

Analysing Eq. (5.2) one can notice that the difference $(w_2 - w_1)$ represents long-lasting deflections due to the creep of the concrete under the permanent loads applied before applying a brittle finish to the floor. Further in this paper these deflections are denoted as w_{creep}. Considering that the approach to the calculation of deflections in [12] is atypical and different than in [11], a diagram (Fig. 7) illustrating this problem was produced. The following two extreme cases were examined:

(a) a disadvantageous case – when a brittle finish is applied to the floor immediately after unpropping ($t = 0$, $\psi = 0$),

(b) an advantageous case – when a brittle finish is applied to the floor a very long time after unpropping ($t > 90$, $\psi = 0.5$).

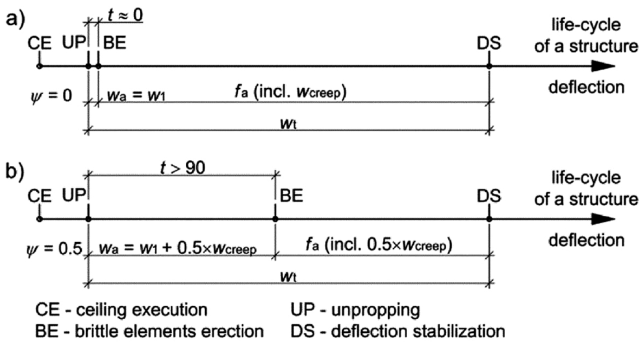

Fig. 7. Diagram for calculating active deflections

It follows from Fig. 7 that when the brittle elements are made immediately after unpropping, one should take the totality of rheological deflections w_{creep} into account in the active deflection (case a). Whereas, when a part of the rheological processes associated with the creep of the concrete partially come to an end (after 90 days), one should take into account only 50% of these effects (case b).

The present authors carried out several calculations and analyses connected with the calculation of deflections in rib-and-block floors with steel truss beams. The deflections of an exemplary 340/600 floor, calculated in accordance with [11] (for a quasi-permanent combination) and [12] (active deflections), are presented in Fig. 8. All the calculations were done for C20/25 concrete, assuming that the time between unpropping and applying a brittle finish is longer than 90 days. The external characteristic load was assumed as equal to 4.5 kN/m^2.

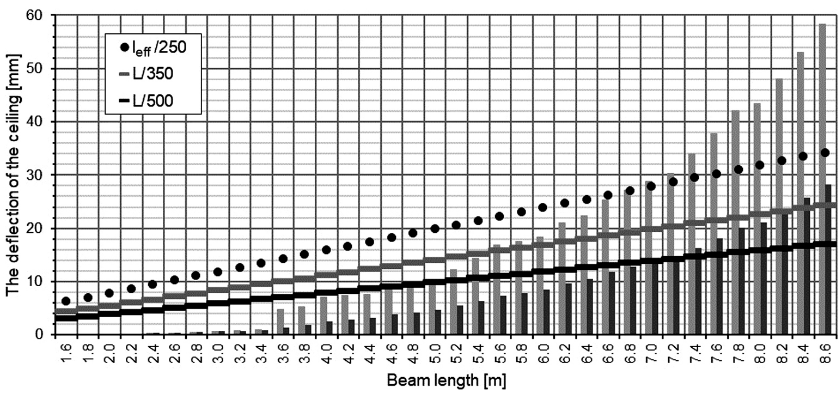

Fig. 8. Floor deflection calculated acc. to [11] at limit $l_{eff}/250$ (light bars) and active floor deflections calculated acc. to [12] at limits L/350 and L/500 (dark bars)

As it appears from Fig. 8, the deflection calculated acc. to [11] exceeds the allowable value for beam lengths of about 7 m. In the case of the calculations acc. to standard [12], active deflection limit L/500 is also reached for beams about 7 m long, but unlike the deflection checked acc. to [11], this active deflection cannot be eliminated or reduced through the use of a pre-camber, which leads to additional difficulties in floor design. When designing rib-and-block floors, it is usually most difficult to meet the active deflection L/500 condition for longer beams. Then the only solution can be to design the floor with continuous multi-span ribs. Another solution to the excessive deflection of the floor can be to make the structure stiffer through the use of, e.g., double beams. The limitation of the active deflection to L/350 is not such a big problem. The allowable deflection for the longest beam was exceeded by about 16%. In this case, only floors with the longest spans may require an individual approach.

Deflection limitation acc. to standard [12] is rather restrictive and so ultimately the designer's experience can be crucial when interpreting the results.

6. Shearing and debonding

The shear ultimate limit state of typical reinforced concrete elements should be checked in accordance with [11]. The checking consists in comparing shear force V_{Ed} with:

– the element's shear capacity, but without taking the shear reinforcement into account, $V_{\mathrm{Rd,c}}$,
– the element's shear capacity, taking into account only the shear reinforcement, $V_{\mathrm{Rd,s}}$,
– the element's shear capacity with regard to the compressed concrete diagonal members, $V_{\mathrm{Rd,max}}$ (mostly decisive for thin-wall elements).

In the case of a floor with beams with steel truss, the shear check should be carried out in accordance to [12] in a way slightly different than the one presented [11]. The checking is carried out for two zones of the cross section and in the contact plane between the two concretes. The line dividing the zones, marked as X-X, is shown in Fig. 9. This line runs from 2 cm (if the strength of the top welds of the truss is equal to the strength of the diagonal members) to 3 cm (if the strength of the top welds of the truss is equal to 60% of the strength of the diagonal members) below the bottom of the top truss bar. Since the ceiling blocks in the considered case are partially structural members, it is allowed to take a 1 cm wide (e = 1 cm) part of the hollow block's wall into account in the calculations.

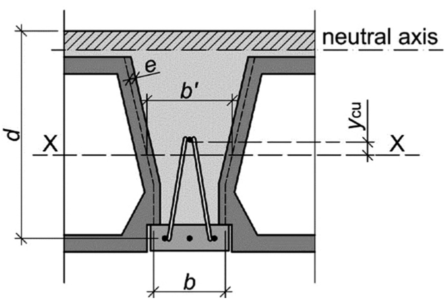

Fig. 9. Cross-section of floor and distances assumed in shear check

As regards shear, the following condition must be satisfied for the upper zone:

$$V_{\mathrm{Ed}} \leq V_{\mathrm{c'u}} = \tau_{\mathrm{cu}} b' z \tag{6.1}$$

where: V_{Ed} – the shear force in the considered cross-section, $V_{\mathrm{c'u}}$ – the shear capacity relating to the upper zone, τ_{cu} – the shear strength of the concrete amounting to $0.03 f_{\mathrm{ck}}$ (f_{ck} – the characteristic shear strength of the concrete), b' – the minimal width in the upper zone with a part of the hollow block taken into account, z – the internal forces' arm amounting to $0.9d$.

As regards the lower zone, the shearing force should satisfy the following condition:

$$V_{\mathrm{Ed}} \leq \max\{V_{\mathrm{cu}}, V_{\mathrm{du}}\} \tag{6.2}$$

Shear capacity V_{cu} relating to concrete is calculated from Eq. (6.3), while V_{du} capacity related to the truss reinforcement is calculated from Eq. (6.4).

$$V_{\mathrm{cu}} = \tau_{\mathrm{cu}} b z, \tag{6.3}$$

$$V_{\mathrm{du}} = \frac{A_{\mathrm{d}} f_{\mathrm{yk}}}{\gamma_{\mathrm{s}}} (\cos\alpha + \sin\alpha) \frac{z}{s_{\mathrm{d}}} + 0.35 f_{\mathrm{ctk\,0.05}} b z \tag{6.4}$$

where: b – the minimal width in the lower zone with a part of the hollow block's wall taken into account, A_{d} – the cross-sectional area of the truss's diagonal member (in the considered case, this is the cross-sectional area of two bars as the diagonal members are situated in two planes), f_{yk} – the characteristic yield point of the reinforcing steel, γ_{s} – the safety factor for the reinforcing steel, amounting to 1.15, α – the angle of inclination of the truss diagonal members to the beam's longitudinal axis, amounting to at least 45°(60°in the considered case), s_{d} – the distance between two parallel diagonal members, measured along the beam's longitudinal axis), $f_{\mathrm{ctk\,0.05}}$ – a 5% quantile of the characteristic tensile strength of the concrete

In addition, considering the contact plane between the two concretes, the shear force should satisfy the following condition:

$$V_{\mathrm{Ed}} \leq V_{\mathrm{wu}} = 2 \frac{A_{\mathrm{d}} f_{\mathrm{yk}}}{\gamma_{\mathrm{s}}} \sin\alpha \frac{z}{s_{\mathrm{d}}} \tag{6.5}$$

It should be noted that besides Eq. (6.5) one should also check debonding resistance (as described below). Moreover, if an additional reinforcement not connected with the diagonal members of the truss was used in the precast beam's flange, one should check whether the shear stresses along the flange thickness do not exceed $0.03 f_{\mathrm{ck}}$. Otherwise a horizontal connective reinforcement should be used.

In the case of the debonding resistance limit state, according to standard [12], one should satisfy the requirements of standard [11] and the additional conditions specified in standard [12]. According to [11], adhesion, the load capacity of the connective reinforcement and the effect of the stresses normal to the contact surface contribute to debonding resistance, which cannot be higher than the appropriately reduced design compressive strength of the concrete ($0.5 \nu f_{\mathrm{cd}}$). According to standard [12], one should additionally check whether the shear stresses on the bond surface are not greater than $0.03 f_{\mathrm{ck}}$. If this condition is not met, one should use a proper connective reinforcement (in the considered case, the diagonal members of the truss act as a connective reinforcement). Then debonding resistance is calculated taking into account only the effect of this reinforcement, but for the diagonal members situated along both the favourable direction and the unfavourable direction, as shown in Fig. 10 (acc. to [11], only the diagonal members situated along the favourable direction are taken into account). The problem of delamination in reinforced concrete structures was discussed in more detail in the article [15].

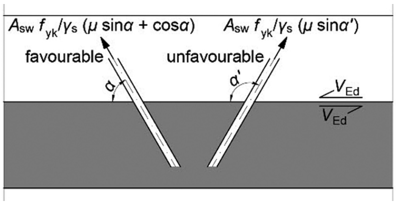

Fig. 10. Principles of determining load capacity of connective reinforcement acc. to [12]

The load capacity of the connective reinforcement acc. to [12] for two diagonal members set at angle α and α' (Fig. 10) relative to the contact surface is calculated from the following relation:

$$F_{\mathrm{Rwd},1} = A_{\mathrm{sw}} \frac{f_{\mathrm{yk}}}{\gamma_{\mathrm{s}}} (\mu \sin\alpha + \mu \sin\alpha' + \cos\alpha) \tag{6.6}$$

where: A_{sw} – the cross-sectional area of the diagonal truss member, α – the angle of inclination of the diagonal truss members situated along the favourable direction (Fig. 10) amounting 45–90°, α' – the angle of inclination of the diagonal truss members situated along the unfavourable direction (Fig. 10) amounting 90–135°, μ – the coefficient of friction.

Exemplary results of the analyses of the shear and debonding problem for floor 340/600 are presented in the bar chart in Fig. 11. The shear capacity values with regard to the lower zone take into account the effect of the truss reinforcement. The debonding capacity values

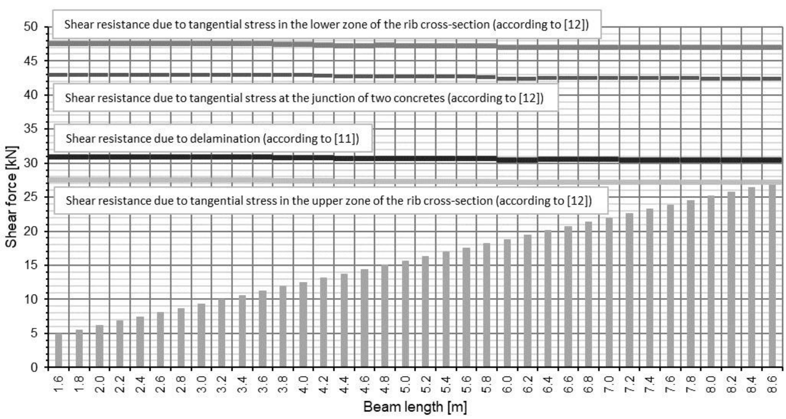

Fig. 11. Shear forces and shear capacity values and debonding resistance

were calculated acc. to [11] as they were dominant (lower) because of this limit state. This is mainly due to the fact that as opposed to standard [12], standard [11] does not enable one to take the connective reinforcement of the diagonal members situated along the unfavourable direction into account.

As it appears from Fig. 11, in the considered case the decisive condition with regard to shear forces is the shear capacity condition stemming from the allowable tangential stresses in the upper zone of the rib's cross section. Because of the very low shear strength of the concrete, amounting to $0.03 f_{ck}$ acc. to standard [12], it is very difficult to meet this condition for larger span floors and floors with a relatively narrow upper zone. A solution in this case can be, e.g., to increase the height of the truss or change the ceiling block geometry. As a result of such measures the minimal cross-sectional width in the upper zone increases and consequently, the tangential stresses decrease.

7. Conclusions

Rib-and-block floor is an effective structural solution which works especially for simple small buildings. By using such floor one can reduce the expenditure of time and work on the building site.

Because of the complicated design guidelines and project completion time pressure floor designers most often use tabulated results, selecting floor components from manufacturer catalogues. The computational analyses reported in this paper were carried out having in mind, i.e., such schedules. The following conclusions emerge from the analyses:

– Because of the need to increase loads from 4.0 to 4.5 kN/m^2 it became necessary to use more reinforcement in the case of some beams.
– By taking into account the share of the reinforcing truss's upper bar in the bending resistance some savings in reinforcement were achieved in the case of some beams, despite the above-mentioned increase in loads (the share of the upper truss bar in the resistance to bending reaches as much as 31%).
– The new guidelines [12] explicitly define the active deflection of a floor. It should be noted that this is a significant supplement to the general information given in [11]. In some cases, in order to meet the restrictive requirements [12] an individual assessment needs to be made and additional stiffening elements need to be introduced into the floor.
– Similarly as in the case of deflections, in order to check shear in accordance with [12] a different approach than the one presented in [11] needs to be adopted. The checking is done in three areas of the cross-section: above the X-X axis (Fig. 9) for the concrete, below the X-X for the truss reinforcement, at the contact between the beam's precast part and its monolithic parts – for the welds or diagonal members of the truss.
– In the design of typical cases the shear ultimate limit state is not decisive.
– Debonding resistance should be checked acc. to [11] and acc. to the additional requirements specified in [12].
– As regards the stress limit state and cracking limit state, standard [12] refers directly to the procedures given in [11].

– Besides the calculation guidelines, standards [12, 13] introduced several structural changes some of which forced substantial changes in the production technology (e.g. a change in the geometry of the ceiling blocks).

Summing up, it should be noted that the introduction of the new standards [12, 13] relating to rib-and-block floors is often not taken seriously and ignored by floor manufacturers and designers, which leads (mainly due to the lack of awareness) to the construction of floors which do not conform to the regulations.

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Analiza stanów granicznych stropu gęstożebrowego na belkach kratownicowych

Słowa kluczowe: ugięcie czynne, stan graniczny, strop gęstożebrowy, żelbet

Streszczenie:

Wśród wielu rozwiązań technologicznych dla stropów w budynkach mieszkalnych oraz niewielkich budynkach użyteczności publicznej bardzo często wybieranym rozwiązaniem są stropy gęstożebrowe belkowo-płytowe. Ich niewątpliwe zalety związane z nieskomplikowaną technologią wykonania, brakiem konieczności stosowania zaawansowanych narzędzi i sprzętu oraz niskim kosztem, są głównym argumentem, który przekonuje inwestorów do wyboru tego typu rozwiązań. Wprowadzenie zharmonizowanych wytycznych europejskich w postaci norm serii EN 15037 spowodowało konieczność wprowadzenia zmian w wymiarowaniu oraz kształtowaniu tych powszechnie stosowanych systemów stropowych, co pociągnęło za sobą konieczność aktualizacji ofert ich producentów. W artykule przedstawiono wymagania wprowadzone przez normy serii EN 15037 oraz główne zmiany wprowadzone w stosunku do wcześniej stosowanych wytycznych i ogólnych obliczeń konstrukcji żelbetowych według Eurokodów. Przedstawiono analizy związane ze stanem granicznym nośności ze względu na zginanie i ścinanie oraz stanami granicznymi użytkowalności. W publikacji wskazano krytyczne punkty w projektowaniu stropów gęstożebrowych belkowo-płytowych oraz przedstawiono wyniki przykładowych obliczeń dla wybranego typu stropu. Dodatkowo przedstawiono i omówiono analizę porównawczą związaną z różnicą zużycia stali zbrojeniowej w wybranym systemie stropowym projektowanym według aktualnych i wcześniejszych wytycznych.

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