Saimaa University of Applied Sciences
Technology, Lappeenranta
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Civil engineering

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DESIGN OF THE COMPOSITE FLOORING SLAB WITH PROFILED SHEETS T-153 (RUUKKI) FOR A RESIDENTIAL BUILDING

Bachelor’s thesis 2015

ABSTRACT

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Design of the composite flooring slab with profiled sheets T153 (Ruukki) for a residential building, 35 pages, 1 appendix

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The purpose of the thesis was to analyse the structure of the composite slab with profiled sheets as permanent formwork; to work with Russian building norms and carry out the limit state design in order to determine characteristics of the structure and find out if it can be used in residential buildings.

The study was commissioned by engineers of company Ruukki rus as they are willing to expand their market from designing industrial buildings to residential apartments. An excel file was created with an intention to suggest constructors a useful tool for the implementation of limit state design of the structure.

The results of the study show that the slab can be successfully used for residential buildings as it meets all the requirements and demands fewer materials.

Keywords: composite slab, reinforced concrete, limit state design, residential construction, profiled sheets.

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1. INTRODUCTION

Area of residential apartments in Russia came to 81 million m^2 according to the data of the Ministry of Construction and Housing utilities. It is 15% more compared to year 2013.

There are more people willing to have their own living space which boosts demand in the real estate market.

Fast assembly, reliability, safety and cost effectiveness are the main priorities of residential construction.

Modern economy allows purchasing any desirable material and technologies allow designing of complex structures for different purposes.

However, building codes do not always provide necessary information and methods of calculating of unstandardized structures.

The aim of this thesis is to study building codes, carry out limit state design, and decide on whether it is possible to use the composite slab in residential buildings or not.

2. COMPANY RUUKKI RUS.

The thesis was made with Ruukki rus. collaboration in Saint-Petersburg. It is a Finnish company established in 1960. In Russia mainly metal frame industrial buildings are produced though the company has an ambition to develop into residential construction.

There is Ruukki's factory in Obninsk, a city near Moscow about 100 km away. It includes three production facilities:

1. Sandwich panels production
2. Heavy metal shop where welding works and painting are done for frames and trusses.
3. Light sheets elements shop where profiled sheets and metal roofing tiles are made.

Profiled sheets are rolled on a special automatically programmed machine. The production line is clean and relatively quiet.

The material is zinc coated thin-walled steel delivered from other factories. The thickness of a sheet varies from 0,8 to 1,5 mm.

Finished sheets are packed in piles and delivered to a client.

3. THE STRUCTURE OF THE SLAB

The composite slabs in question consist of profiled steel decking with reinforced concrete on top. (Figure 1)

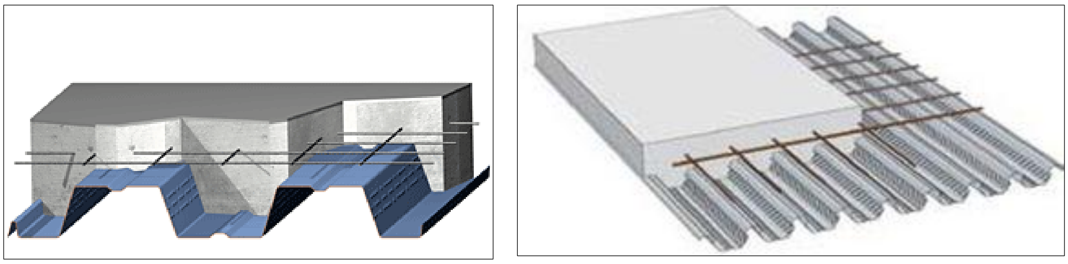

Figure 1 Visual representation of a composite slab

Reinforced concrete itself is a composite material. It became popular in the early 1900 in the USA for the first skyscrapers because it is lighter and cheaper than solid steel elements. It is well known that concrete is efficient in compression and steel is in tension. Together they manifest the most efficient ratio of cost and performance.

Construction industry is constantly searching for faster production and cheaper structures. Reinforced concrete with profiled sheets as permanent shuttering is exactly such an example.

3.1. The structure

The flooring composite slab is made of concrete, steel reinforcement and profiled sheets T-153 (Ruukki) serving as a permanent formwork. The slab rests on steel T-beams with flanges oriented in the bottom. The height of the slab is 220 mm.

The structure acts at two stages. At the first stage (Construction) the loads of poured concrete, profiled sheets, workers, machinery and reinforcement are carried by the profiled shuttering. At the second stage (Operating) payloads and loads of slabs’ self-weight and partitions are carried by the slab itself.

To create a finished look of the ceiling it is suggested that two layers of gypsum plasterboard are fastened to the profiled sheets by metal furring channels and bolts.

The main source of information for the algorithm of calculations was STO 0047-2005 «ZNIIPSK Melnikova ltd», «Hilti Distribution ltd». Also SNiPs and GOSTs were used to conduct the study.

Calculations were carried out to derive:

1. Strength and deflection of the profiled sheets during the erecting
2. Reinforcement diameter
3. Strength of inclined sections
4. Bearing stress of the slab on the supports
5. Deflections of the slab

Anchorage of the profiled sheets is conducted if the sheets are considered as a bearing structure during the operating stage.

According to the fire safety requirements fire resistance of bearing structures in residential buildings is R90 which means that in case of fire the structure has to keep its bearing ability during 90 minutes.

Steel possesses high level of thermal conductivity, its fire resistance is R10-R15, critical temperature (before steel reaches yield limit) is about 500 °C. Such characteristics are not suitable for residential buildings therefore the profiled sheets cannot be considered as the bearing structure at the operating stage if only special fire protective layer is added. However, it is not cost effective.

Based on that explanation anchorage of the profiled sheets can be neglected.

3.2. Conditions

According to STO 0047-2005 there are a few suitable conditions for the use of the composite slab:

1. Nonaggressive/mildly aggressive environment (no harmful gases)
2. Humidity less than 75%
3. Temperature less than +30℃
4. Concrete should not contain chlorides
5. Minimum fire resistance RE30

These conditions are compatible with requirements for residential buildings.

According to GOST 27751-88 “Reliability of constructions and foundations” residential buildings should be referred as structures with the second (normal) level of reliability. Then the reliability coefficient γ_n=0,95.

3.3. Materials

In this project lightweight concrete is accepted. B12.5 is the minimal required strength class. (STO 0047-2005)

Coiled steel is used for profiled sheets.

Steel rods A-III(A400) and steel wires Bp-I are used as reinforcement.

The slab is positioned on steel T-beams which can be rolled or compound.

3.4. Construction requirements

The minimal thickness of the concrete layer above the profiled sheet should be 30 mm according to the fire safety requirements. If screed is not used, then the concrete layer is 50 mm. (STO 0047-2005)

Profiled sheets are overlapped and joined together by screws or rivets. They can be adjusted to the T-beams by screws or nails.

STO 0047-2005 suggests that the length of the support of the slab should be at minimum 40 mm and the minimum thickness of the profiled sheets should be 0,7 mm. The length of the support is accepted 90 mm in the calculations, based on the recommendations of an experienced engineer.

3.5. Corrosion

There is nonaggressive environment in residential buildings, humidity is 45-75%. However, in bathrooms and kitchens humidity can vary, therefore the slab has to be protected additionally.

3.6. Fire resistance

Multi-storey residential buildings have the second degree of fire resistance meaning that bearing structures are nominated with R90 fire endurance.

In case of fire a concrete cover protects reinforcement from collapse. According to SP 52-101-2003 “Concrete and reinforced concrete structures without prestressing” the minimum height of the concrete cover for the main reinforcement should be 20 mm for indoor structures with normal humidity. For secondary reinforcement the concrete cover should be at least 15 mm.

Therefore the wider corrugation (120 mm) is placed in the bottom to provide necessary concrete protection for steel bars (formula 2, 3).

Hence, the depth of concrete cover for the main reinforcement is accepted 30 mm and 15 mm for secondary upper reinforcement.

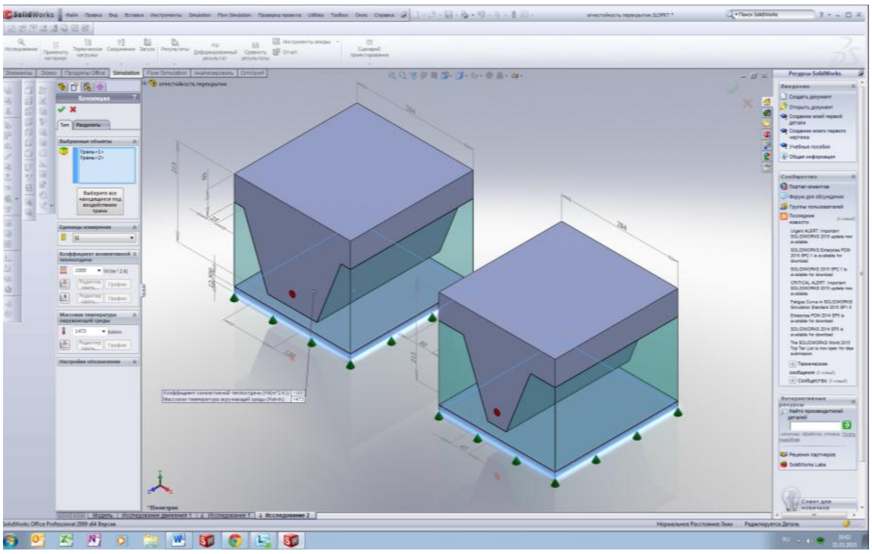

Figure 2 Different placement of corrugations of the slab

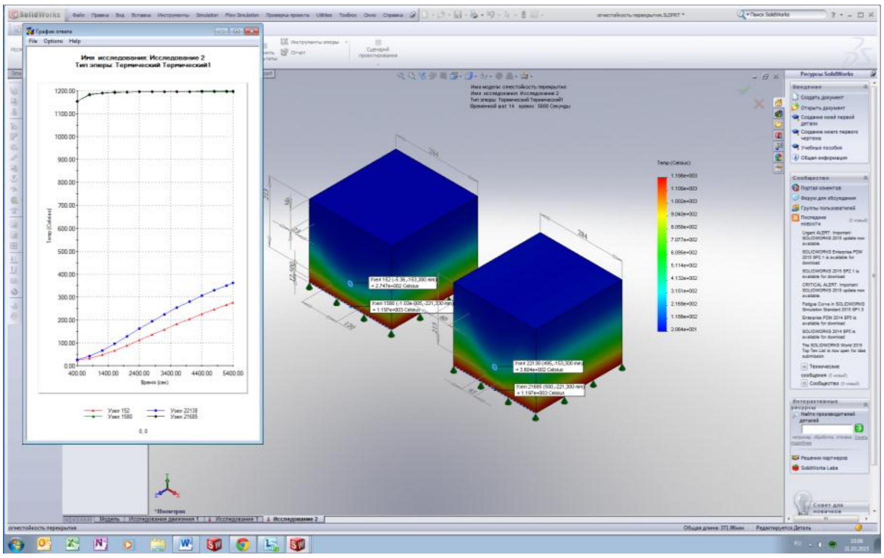

Figure 3 Heat distribution in the slabs with different positions of corrugations

3.7. Acoustics

Aerated concrete reduces impact noise and background sound. Therefore it is a perfect material for residential buildings. The required sound insulation index is 54 dB. An additional sound proofing layer of velimat 4 mm will be used. Its sound insulation index is 29 dB.

3.8. Floor layers

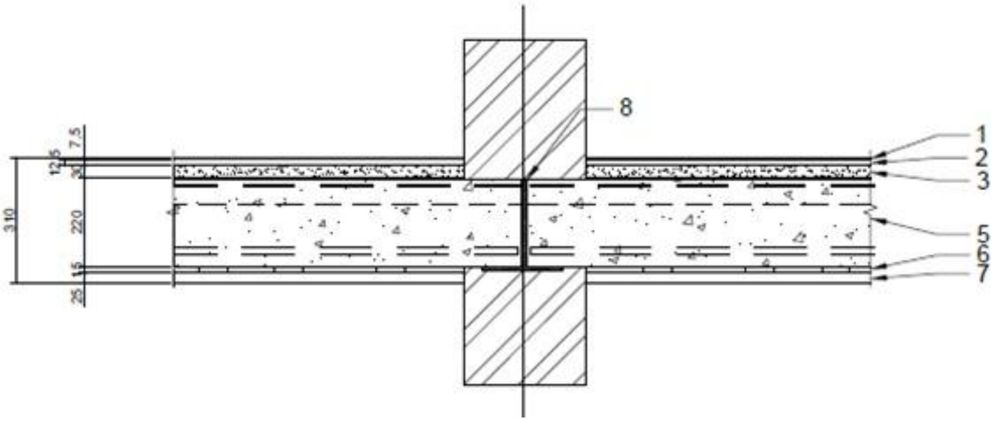

Figure 4 A cross-section of the slab

Table 1 Flooring layers

№ in the figure	Layer	Height	Unit
1	Floor covering (carpet, laminate, etc.)	7,5	mm
2	Gypsum fibre board	12,5	mm
3	Screed	30	mm
4	Velimat (sound proofing layer)	4	mm
5	Composite slab	220	mm
6	Metal furring channels	15	mm
7	Gypsum plasterboard (2 layers)	25	mm
	Total	314	mm

4. CONSTRUCTION STAGE

During the construction period, when concrete is liquid and has not achieved cube strength yet, the profiled sheets are considered as bearing structures. It is necessary to derive strength and deflection for the sheets as for a thin-walled element which bears its own weight, weight of reinforcement, concrete and the erection load (workers and machinery).

4.1. Characteristics of the profiled sheet T153-120L-850

The material of the profiled sheets is hot dipped galvanized cold rolled steel type C320.

The length can vary from 2000 to 12000 mm.

The available thickness is 0.8; 1; 1.2; 1.5 mm

«T» means that corrugations have trapezoidal form

«153» means that the height of the sheet is 153 mm.

«120» or «43» means that the width of top or bottom webs is 120 or 43 mm respectively.

«L» indicates a bearing sheet

«850» means that the module width is 850 mm.

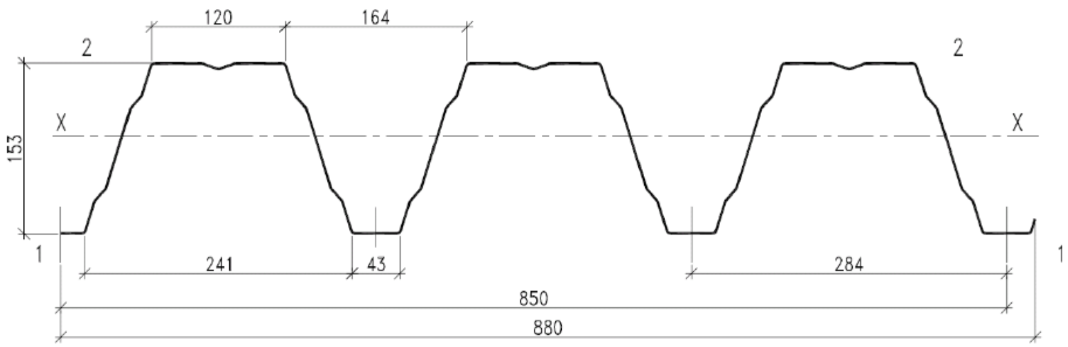

Figure 5 Cross-section of profiled decking

4.2. Strength calculation

The height of the sheet is 153 mm.

The height of the T-beam's web is 220 mm.

Consequently, the depth of concrete above the profiled sheet is derived by formula 1

$$h_f = H - h_n \qquad (1)$$

$$h_f = 0{,}22 - 0{,}153 = 0{,}067\text{m}.$$

The effective height of the composite slab is derived by formula 2

$$h_b = h_f + \frac{(b + b')}{2s_0} h_n \qquad (2)$$

where

h_f - concrete depth above the profiled decking

b - width of the bottom flange (corrugation) of the sheet

b'- width of the top flange (corrugation) of the sheet

h_n – height of the profiled decking

s_0 – space between centres of the nearest flanges of the profiled decking

$$h_b = 0{,}067 + \frac{(0{,}120 + 0{,}241)}{2 * 0{,}284} 0{,}153 = 0{,}164 \text{ m}$$

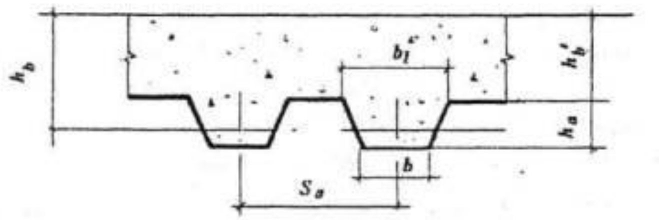

Figure 6 Cross section of the slab

The trapezoidal cross-section with width b_f=284mm is used in the calculation

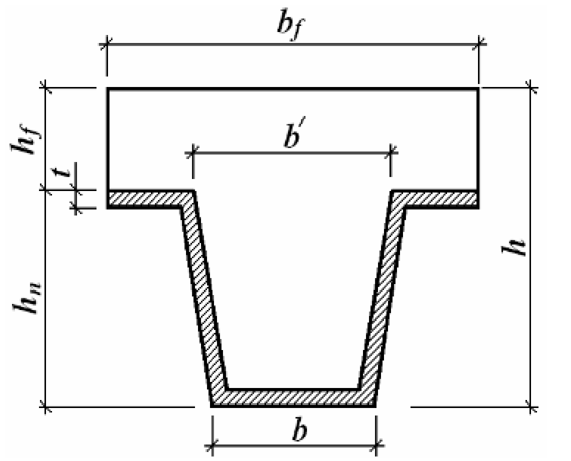

Figure 7 Trapezoidal cross-section of the slab

As an analytical diagram one span beam is accepted because the sheets are not tied together across supports and work separately.

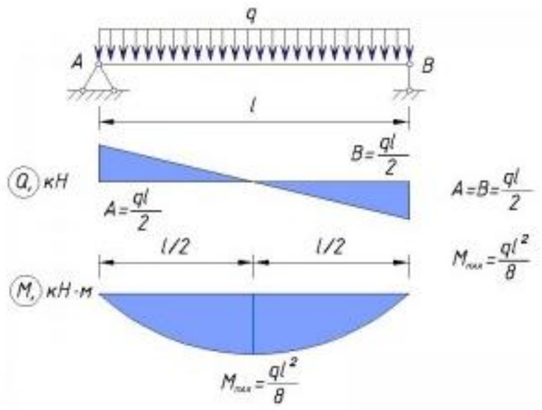

Figure 8 One span analytical diagram

Formula 3 is used to check whether strength is ensured or not.

$$\sigma = \frac{M}{W} \leq [\sigma] = \frac{R_y}{\gamma_n} \quad (1)$$

where

σ –stress in compressed area;

$M = \frac{ql^2}{8}$ –bending moment;

R_y –design value of tensile resistance

γ_n – safety factor;

W=0,0000431 m^3 resistance moment per one metre.

$$M = \frac{ql^2}{8} = \frac{3.44 * 3.5^2}{8} = 5.26 \text{ kN/m}$$

$$\sigma = \frac{5,26}{0,0000431} = 122087,33 \text{ kN/m}^2$$

$$[\sigma] = \frac{R_y}{\gamma_n} = \frac{312,20 * 1000}{0,95} = 328626,44 \text{ kN/m}^2$$

The maximum allowed stress is three times bigger than the calculated stress meaning that strength is ensured definitely.

4.3. Deflection calculation

Characteristic loads are used to derive deflection (formula 4)

$$f_n = k\frac{q^n l^4}{E_a J_x} \leq [f] = \frac{1}{200} l \qquad (2)$$

where

f_n – maximum deflection

k=5/384 – coefficient, determined according to analytical diagram type;

q^n- characteristic load;

l – span length;

E_a – elasticity modulus of the profiled sheet;

J_x – moment of inertia per one metre.

$$f_n = k\frac{q^n l^4}{E_a J_x} = \frac{5}{384} * \frac{2,68 * 3,5^4}{21 * 10^7 * 3,985 * 10^{-6}} = 0,006 \text{ m}$$

$$[f] = \frac{1}{200} l = \frac{1}{200} * 3,5 = 0,0175 \text{ m}$$

The calculated deflection does not exceed the maximum allowed deflection. Consequently, the profiled sheet T153 with the 0.8 mm thickness – the smallest thickness available - and steel type C320 is accepted as it satisfies the requirements of strength and deflection.

5. OPERATING STAGE

During the second stage the composite slab is considered as the bearing structure. The profiled sheets are considered as permanent formwork, they do not bear loads.

5.1. Design of the composite slab

Limit state design is carried out to derive whether the slab meets the requirements of strength and deflection or not.

For the calculations the following assumptions are accepted:

1. Tensile strength of concrete equals zero.
2. Stresses in the profiled sheets are evenly distributed along the height and equal design value of resistance of steel R_y=312,20 N/mm^2 considering service factor γ_c=0,8
3. Stresses in the main reinforcement equal design compression resistance R_{sc}=355 N/mm^2 and tensile resistance R_s=355 N/mm^2 considering adequate service factors
4. The effective depth h_0 is the height of the reinforcing steel from beginning of the reinforced concrete section in compression to the reinforcing steel bars in tension

For the study the composite slab is considered as a two-span continuous hinged beam because the slabs are assumed to work together by steel rebars between supports.

5.2. Reinforcement area derivation

The profiled sheet is not a bearing structure at the operating stage, therefore it is considered only as permanent formwork. It cannot also be additional reinforcement because there is not a strong connection between the sheets and concrete.

Rebars of the main reinforcement are placed longitudinally in each corrugation of the sheets. Steel wires are welded to the top and bottom reinforcement creating a dimensional grid.

For the steel type AIII (A400) design tensile resistance of longitudinal and lateral reinforcement is f_{yd}=355 N/mm^2

Modulus of elasticity for steel is E_S=2*10^5 N/mm^2

The concrete cover of the top reinforcement is 15 mm and 30 mm of the main reinforcement according to the fire safety requirements.

The limit value of relative compression zone depth is derived by formula 5

$$\zeta_r = \frac{\omega}{1 + \frac{R(1 - \frac{\omega}{1,1})}{\sigma_{sr}}} \quad (3)$$

where

ω – factor of stress diagram;

ω = 0,8 - 0,008*Rb – formula for aerated concrete;

R – reinforcement stress with allowance for reinforcement yield limit

σ_{sr} - limit stress in compression reinforcement. For reinforced concrete (without prestressing) if $f_{yk} \leq$ 400 N/mm^2 then $\sigma_{sr} = f_{yd}$ =400 N/mm^2 is accepted;

$$\zeta_r = \frac{0,718}{1 + \frac{355(1 - \frac{0,718}{1,1})}{400}} = 0,549$$

In case of a collapse, the compression zone is not allowed to break down because it crushes suddenly and unexpectedly. Tensile zone breaks down gradually. Firstly, cracks appear and there is a possibility to eliminate defects and avoid risks without any victims.

Therefore the maximum compression zone height is

$$x = \zeta_r h_0 \quad (4)$$

$$x = 0{,}540 * 0{,}19 = 0{,}103 \text{ м}$$

$$x > h_f$$

Compression zone depth is bigger than the effective height of the slab which indicates that formulas 7and 8 should be used.

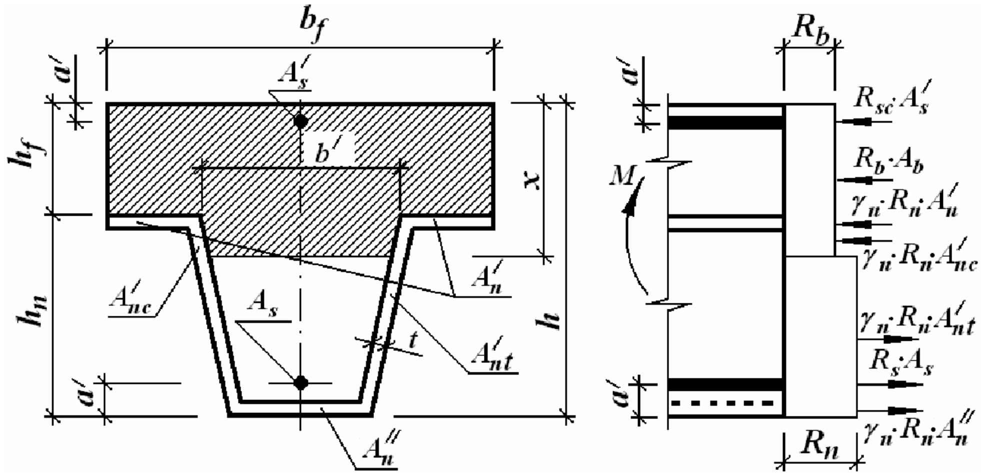

Figure 9 Stress distribution

$$R_b[(b'x + (b_f - b')h_f] + R_{sc}A'_s = +R_sA_s \quad (5)$$

$$M_{span} \leq R_bS_{bx} + R_sS_{sx} + R_{sc}S'_{sx} \quad (6)$$

where

M_{span} – bending moment

S_{bx} – first moment of compression zone area

S_{sx} , S'_{sx} - first moments of tensile and compression reinforcement accordingly.

The first moments are calculated geometrically.

Hence:

A_s= 219 mm^2 – area of tensile reinforcement

A'_s= -570 mm^2 – area of secondary reinforcement

For bottom reinforcement steel type AIII (A400) rods Ø18 cm^2 in each corrugation are accepted.

The area of upper reinforcement is negative which means that it is not necessary for bearing purposes. However, it should be installed because the steel grid will prevent concrete from setting shrinkage and from spalling. Therefore the upper reinforcement made of steel type Bp-I Ø4 mm with spacing 200x200 mm is accepted.

For a comparison, concrete type B25 was considered. It possesses higher density – 2200 kg/m^3, while concrete B12,5 has 1200 kg/m^3. Hence, the overall load almost doubles: from 484 kg/m^2 to 784 kg/m^2 only because of increased self-weight. Therefore another diameter of reinforcement is needed - Ø20 mm in each corrugation.

5.3. Steel rebars between supports

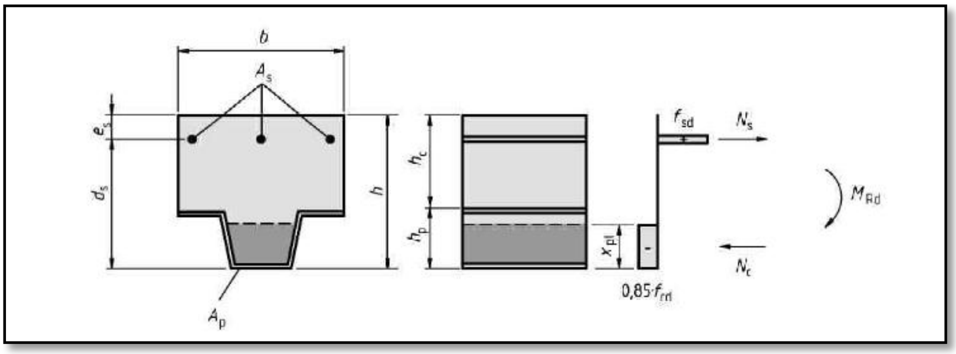

Figure 10 Stress distribution of negative moment

The rods are placed between supports of the slabs. The idea is to create an I-beam section from a T-beam and a rod combining by concrete. Such a structure allows replacing of the top flange by concrete partially which makes a considerable cost saving, besides, it simplifies adjusting of the profiled sheets to the beam by screws.

The negative moment Mв=7,84 kNm acts in the middle support, therefore the compression zone of concrete moves to the bottom part of the section. It is necessary to derive the diameter of the top (tensile) reinforcement rods for this case.

$$N_c = N_s \tag{7}$$

$$A_s R_s = A_c R_b$$

$$A_s = \frac{A_c R_b}{R_s} = \frac{R_b}{R_s} * \frac{(b + b^{'})}{2} h_n$$

$$A_s = \frac{10000}{355000} * \frac{(0{,}120 + 0{,}241)}{2} * 0{,}153 = 0{,}00079\ м2$$

To simplify the calculation the web of the section is considered as a compressed zone.

$$A_s = 7{,}9\ см^2$$

The result is way too big for the size of the section.

A different type of an analysis can be conducted to solve the issue: the diameter of the top steel rods can be derived based on the assumption that in case of bending the section undergoes oblique shear forces. Then the middle support reaction equals the sum of design loads on the slab divided by the sum of area of slab supporting and area of longitudinal section of a steel rebar

$$B = \frac{q^p}{nS_{ст} + A_{loc}} \qquad (8)$$

Where

B- middle support reaction;

q^p- design load at the operating stage;

n- number of steel rods in the flange of the section;

$S_{ст} = l_{ст}D$- area of longitudinal section of a reinforcement rod, going through the middle of the rod.

$l_{ст}$- half of the rod length

D- diameter of the steel rod

$A_{loc} = bd$ – bearing area on the T-beam

b- width of the bottom part of corrugation d- length of a corrugation bearing on the beam

$$nD = 1{,}21 \text{ m}$$

If n=1, then D=1,21 m

If n=2, then D=0,6 m

Such values of diameters cannot be implemented in the slab in question.

Based on the results there is an assumption that the stress diagram (continuous beam) is chosen incorrectly.

Tests, conducted in a laboratory, show (Figure 11) that in case of collapse the top reinforcement rods do not work with concrete together. Concrete breaks down faster and the rods are not pulled out from the structure. Consequently, the slabs work independently from each other and it is vital to consider them as one-span beams.

Figure 11 Laboratory tests

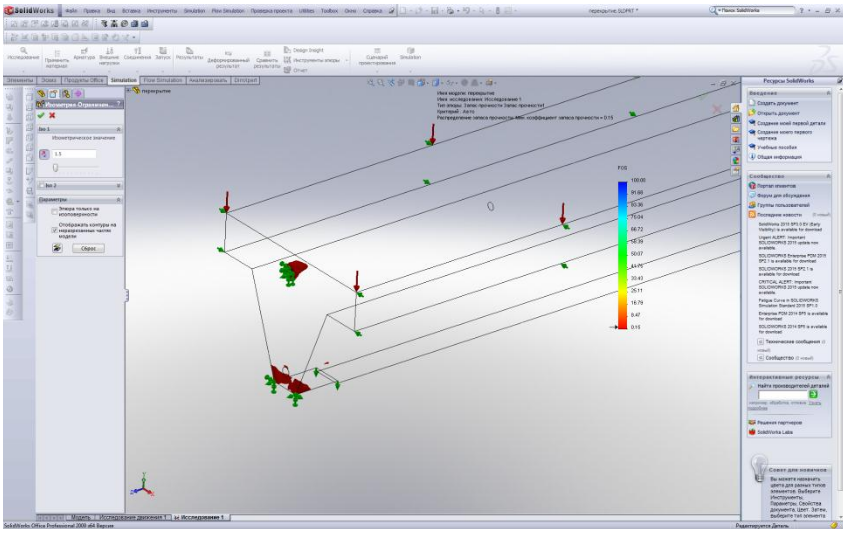

Figure 12 Stresses on the supports

5.4. 5.4. Inclined sections strength under shear force action

This calculation is carried out for the two stages of work of the slab.

SP 52-01-2003 recommends that the diameter of stirrups in bending structures should be at minimum 6 mm.

Therefore stirrups with diameter 6 mm and spacing 150 mm are accepted.

STO 0045-2005 suggests that the angle of the inclined crack is 45̊ (Figure 13)

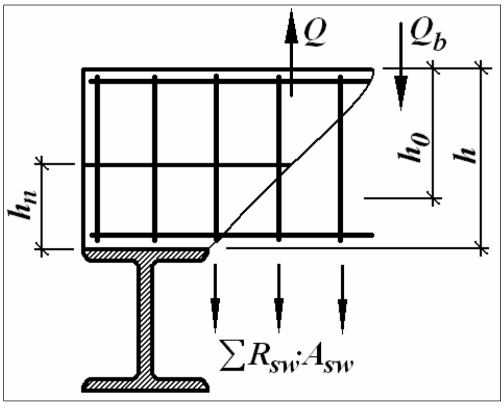

Figure 13 Inclined section stress distribution

Two conditions must be met (formula 11)

$$\begin{cases} Q \le 0{,}17R_y\gamma_c h_n 2t + \sum R_{sw}A_{sw} + Q_b \\ Q \le 0{,}3\varphi_{W1}\varphi_{b1}R_b(b + \frac{b'}{2})h_0 \end{cases} \quad (9)$$

Where $0{,}17R_y\gamma_c h_n 2t$ – lateral stress which appears in a profiled sheet in one corrugation

$\sum R_{sw}A_{sw}$- sum of lateral stresses appearing in stirrups which cross inclined section

Q_b- lateral stress in concrete

$\varphi_{W1} = 1{,}16$ - coefficient according to SNiP 52-01-2003.

$\varphi_{b1} = 0{,}796$ – coefficient according to SNiP 52-01-2003.

The lateral stress Q_b is derived by formula 12

$$0{,}5R_{bt}h_0\left(b + \frac{b'}{2}\right) \le Q_b = \frac{\varphi_{b2}R_{bt}\left(b + \frac{b'}{2}\right)h_0^2}{c} \le 2{,}5R_{bt}h_0\left(b + \frac{b'}{2}\right) \quad (10)$$

$$Q_b = \frac{1{,}5 * 6{,}4 * (0{,}120 + 0{,}241) * 0{,}019^2}{0{,}22} = 378{,}85 \text{ kN}$$

where

c- length of projection of the inclined section onto a longitudinal axis of the element.

The angle of the crack is accepted 45°, therefore c=h

φ_{b2}- coefficient 1,5

The inclined sections’ calculation was carried out for the two stages:

<u>1 Stage: Construction</u>

$$Q \le 0{,}17R_y\gamma_c h_n 2t \quad (11)$$

$$0{,}17R_y\gamma_c h_n 2t = 0{,}17 * 312200 * 0{,}8 * 0{,}1538280{,}0008 =$$
$$= 10{,}39\ \mathrm{KN}$$

$$Q = \frac{3ql}{8} \qquad (12)$$

$$Q = \frac{3 * 3{,}44 * 3{,}2}{8} = 4{,}2\ \mathrm{kN}$$

4,2kN<10,39 kN

The strength requirement is met.

<u>2 stage: Operating.</u>

$$\begin{cases} Q \le \sum R_{sw}A_{sw} + Q_b \\ Q \le 0{,}3\varphi_{W1}\varphi_{b1}R_b(b + \frac{b'}{2})h_0 \end{cases} \qquad (13)$$

$$Q = \frac{3 * 6{,}79 * 3{,}2}{8} = 27{,}15\mathrm{kN}$$

$$\sum R_{sw}A_{sw} + Q_b = 285000 * 0{,}006 + 378{,}85 = 386{,}01\ \mathrm{kN}$$

Only one stirrup is in the section.

$$0{,}3\varphi_{W1}\varphi_{b1}R_b\left(b + \frac{b'}{2}\right)h_0 = 0{,}3 * 1{,}16 * 0{,}796 * 10200 * \left(0{,}120 + \frac{0{,}241}{2}\right) * 0{,}019 =$$
$$= 128{,}78\ \mathrm{kN}$$

$$\begin{cases} 27{,}15\ \mathrm{kN} \le 386{,}01\ \mathrm{kN} \\ 27{,}15\ \mathrm{kN} \le 128{,}78\ \mathrm{kN} \end{cases}$$

The condition is met, consequently, the strength of inclined sections is provided

5.5. Local compression strength

Supports of the slab are checked for local compression strength. Condition 16 must be met

$$N \le 0{,}5R_bA_{loc} \qquad (14)$$

where

N – support reaction in one corrugation

Aloc – area of compression (formula 17)

$$A_{loc} = ba \qquad (15)$$

where

b- width of the bottom flange of the support

a=0,09m – length of the supporting of the slab on the beam

$$N = \frac{3q(l-2a)}{8} = \frac{3 * 6,79 * (3,5 - 2 * 0,09)}{8} = 8,49 \text{ kN}$$

$$0,5R_bA_{loc} = 0,5 * 10,2 * 1000 * 0,09 * 0,12 = 55,08 \text{ kN}$$

The condition is met.

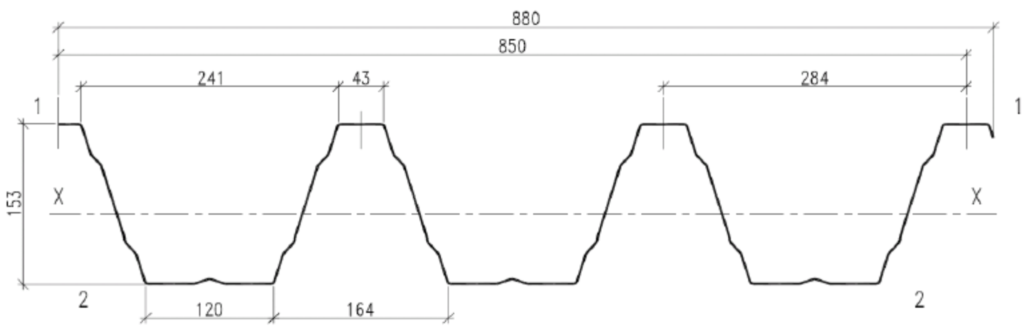

Figure 14 A cross-section of the profiled sheet

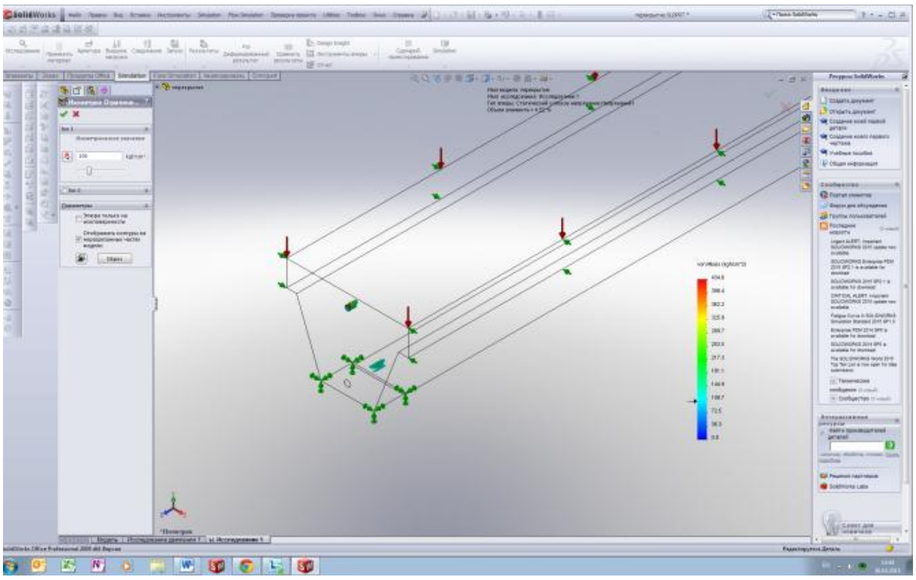

Figure 15 A model of local compression

5.6. Deflection of the composite slab

Deflection of the slab is derived by the sum of deflections of the profiled sheets during the construction phase and deflection of the concrete slab.

In the calculation characteristic loads are used. The calculated deflection should not exceed the maximum value (formula 18,19)

$$f \leq f_{ult} \tag{16}$$

$$f \leq \left[\frac{1}{150}\right] l \tag{17}$$

where f – deflection of the slab caused by the loads at the operating phase.

$f_{ult} = \left[\frac{1}{150}\right] l$ – maximum tolerable deflection of the slab.

The slab spans work independently from each other, therefore on-span beam is used as the stress diagram for calculations

The method is described in SP 52-101-2003

$$f = sl^2 \left(\frac{1}{r}\right)_{max} \tag{18}$$

where

s-5/48- coefficient depending on the stress diagram and the type of a load.

$\left(\frac{1}{r}\right)_{max}$ - maximum curvature in the section with the maximum bending moment from the load

$$\left(\frac{1}{r}\right)_{max} = \left(\frac{1}{r}\right)_1 + \left(\frac{1}{r}\right)_2 \tag{19}$$

$\left(\frac{1}{r}\right)_1 + \left(\frac{1}{r}\right)_2$-curvatures of short term loads and long term impact of permanent loads

$$\left(\frac{1}{r}\right) = \frac{M}{E_{b1}J_{red}} \tag{20}$$

where E_{b1}- deformation modulus of compressed concrete depending on duration of the load.

For short term loads formula 23 is used

$$E_{b1} = 0{,}85E_b \tag{21}$$

$$E_{b1} = 0{,}85 * 10000000 = 8500000\ \mathrm{kN/m^2}$$

For long term loads formula 24 is used

$$E_{b1} = \frac{E_b}{1+\varphi_{b,cr}} \tag{22}$$

$$E_{b1} = \frac{10000000}{1 + 3{,}65} = 2150638\ \mathrm{kN/m^2}$$

$\varphi_{b,cr}$=3,65- creep coefficient for concrete B 12,5

J_{red}- moment of inertia of the effective cross-section relatively its centre of gravity (formula 25)

$$J_{red} = J + J_s a + J_s' a \tag{23}$$

J – moment of inertia of effective cross-section relatively centre of gravity derived considering whether there are cracks in concrete or not.

J_s и J_s' - moments of inertia of section area of tensile and compressed reinforcement relatively the centre of gravity of the reduced section of an element.

a - coefficient of correlation between concrete and reinforcement.

SP 52-101-2003 allows deriving of the moment of inertia J_{red} excluding reinforcement.

Another toleration is that the moment of inertia of the profiled sheets can be neglected as well

$$J_{red} = J_y = \frac{b_f h_f^3}{12} + \frac{b h_n^3}{12} + b_f h_f (y_0 - \frac{h_f}{2})^2 + b h_n (\frac{h_n}{2} + h_f - y_0)^2 \tag{24}$$

$$y_0 = \frac{b_f h_f^2 + b h_n (2h_f + h_n)}{2(b_f h_f + b h_n)} \tag{25}$$

$$J_{red} = 0{,}000156 m^2$$

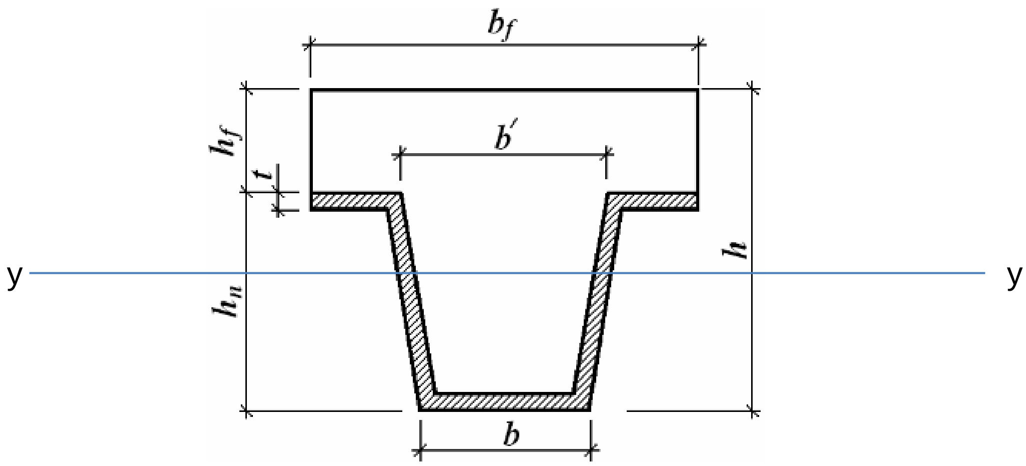

Figure 16 A trapezoidal cross-section of the slab

Payload – 1,5 kN/r.m.is considered as short term characteristic load for the calculation.

The moments are derived by formula 28:

$$M = \frac{q^{н} l^2}{8} \tag{26}$$

$$M^{\text{short term}} = \frac{1{,}5 * 3{,}2^2}{8} = 2{,}43 \text{ kNm}$$

$$M^{\text{long term}} = \frac{3{,}76 * 3{,}2^2}{8} = 6{,}09 \text{ kNm}$$

The curvature equals:

$$\left(\frac{1}{r}\right)_{max} = \frac{2{,}43}{8500000 * 0{,}000156} + \frac{6{,}09}{2150538 * 0{,}000156} = 0{,}019$$

Deflection of the slab

$$f = \frac{5}{48} * 3{,}6^2 * 0{,}02 = 0{,}027 \text{ m}$$

$$\left[\frac{1}{150}\right] l = \frac{3{,}6}{150} = 0{,}024 \text{ m}$$

The derived deflection exceeds the maximum deflection. The simplest and more economically effective solution is to limit the span length of the slab.

If span length is 3,2 m. deflection f=0,017 m. Then the maximum allowable deflection is $\left[\frac{1}{150}\right] l = 0{,}021$ m.

The condition $f \leq \left[\frac{1}{150}\right] l$ is met.

Chart 1 Deflection of the slab

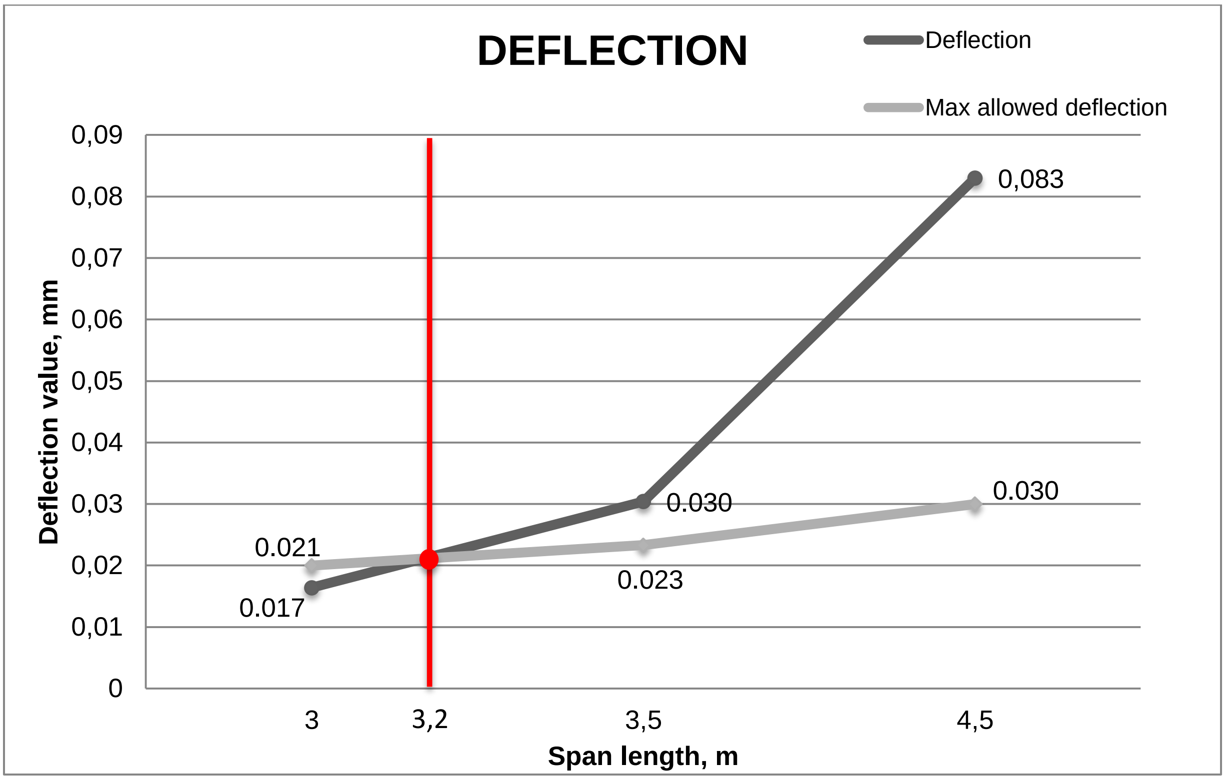

Chart 1 reflects the dependance of deflection from span length. In order to keep with the requirements it is clear that the length of the span should be limited to 3,2 m.

6. THE EXCEL PROGRAMME

The excel programme was made with the purpose to create a simple, helpful tool allowing an engineer to choose and compare different variants of the structure in question.

To undertake the calculations the programme requires complete information about materials and geometric characteristics of the structure. The programme then automatically creates a table of loads: for the construction and operating stages. (Table 2)

Table 2 Table of loads

№	Load stages	Name	Density	Thickness	Characteristic loads, qn	Characteristic loads per a running metre, qn	γf - safety factor for loads	Design loads,qp	Design loads per a run metre, qp
			kg/m3	m	kg/m2	kN/r.m.		kg/m2	kN/r.m.
1	2	3	4	5	6	7	8	9	10
1	I stage-Construction loads	RC slab self weight	1200	0.164	197.09	1.97	1.3	256.22	2.56
2		Profiled sheet self weight		0.0008	11.50	0.12	1.05	12.08	0.12
3		Steel rebars between the supports			9.85	0.10	1.05	10.35	0.10
4		Construction load from workers and machinery			50.00	0.50	1.3	65.00	0.65
	Total				**268.44**	**2.68**		**343.64**	**3.44**
1	II stage-Operating loads	RC slab self weight	1200	0.164	197.09	1.97	1.3	256.22	2.56
2		Profiled sheet self weight		0.0008	11.50	0.12	1.05	12.08	0.12
3		Steel rebars between the supports			9.85	0.10	1.05	10.35	0.10
4		Floor covering (carpet, laminat	900	0.0075	6.75	0.07	1.3	8.78	0.09
5		Gypsum fibre board	1200	0.0125	15.00	0.15	1.3	19.50	0.20
6		Screed	2200	0.03	66.00	0.66	1.3	85.80	0.86
7		Walls and partitions			50.00	0.50	1.3	65.00	0.65
8		Plaster slabs (2 layers)			20.00	0.20	1.3	26.00	0.26
9		Payloads			150.00	1.50	1.3	195.00	1.95
	Total				**526.19**	**5.26**		**483.71**	**6.79**

The next page determines whether the chosen profiled sheet is suitable for bearing of the loads or not. (Table 3, 4).

Table 3 Strength of the profiled sheets

Bending moment M=ql2/8	Moment of resistance, W	Calculated strength, σ=M/W	Condition σ<=[σ]	Maximum allowed strength,	Conclusion
kNm	m3	kN/m2		kN/m2	
4.40	0.0000431	102055.04	<	328626.44	*Correct*

Table 4 Deflection of the profiled sheets

Characteristic loads, qn	Module of elastisity for steel, Es	Moment of inertia of the profiled sheets, Ix	Deflection, f	Condition f<=[f]	Maximum allowed deflection [f]=l/200	Conclusion
kN/m2	kN/m2	m4	m		m	
2.68	210000000	0.000003985	0.004	<	0.016	*Correct*

The programme also defines the area of reinforcement, the strength of inclined sections, the strength on the supports and the deflection of the slab. (Table 5,6,7)

Table 5 Strength of inclined sections

Stage	Stress caused by Q	Condition	Value of the formula	Formula	Conclusion
	кН		кН		
I	4.12	<	10.39	$Q \leq 0{,}17 R_y \gamma_c h_n 2t$	*correct*
II	27.15	<	386.91	$Q \leq \sum R_{sw} A_{sw} + Q_b$	*correct*
	27.15	<	128.78	$Q \leq 0{,}3 \varphi_{w1} \varphi_{b1} R_b (b + \frac{b'}{2}) h_0$	*correct*

Table 6 Bearing stress on the supports

Strength	Reaction, N	Condition	0,5*Rb*Aloc	Conclusion
	kN		kN	
On the supports	7.69	<	55.08	*correct*

Table 7 Deflection of the slab

Deflection of the slab	Summed deflection	Value	Limit deflection, [1/150]*l	Conclusion
m	m		m	
0.0169	0.0212	<	0.0213	*correct*

7. CONCLUSIONS

There are remarkable results obtained:

1. Wider corrugations of the profiled sheets should be placed in the bottom to meet the local compression strength requirements and to create enough concrete covering for working reinforcement.
2. The span of the slab should be limited to 3200 mm in order to satisfy the deflection requirements.
3. The slabs do not work together even though they are combined y steel rods between supports, therefore, a one span diagram should be considered in calculations.
4. The flooring system requires fewer materials (concrete and steel), therefore it is lighter than the regular reinforced concrete slabs and applies less loads to lower layers which is truly beneficial when speaking of multi-storey developments.

8. SUMMARY

The purpose of the thesis was to analyse the flooring system – the composite slab – with the profiled sheets T153-850-L-150 (Ruukki) as permanent shuttering; to carry out limit state design and determine whether the structure is suitable for residential buildings or not; to create an Excel programme aiming at helping an engineer to calculate and compare different characteristics of the slab quickly.

The results can be assessed as satisfying. The composite slab can be efficiently used in high-rise developments, besides, it has benefits over other flooring systems as there is no necessity for shores during erection which are aimed at supporting formwork loads and liquid concrete until concrete becomes structurally self-sufficient. Contractors do not need any kind of supports because the profiled sheets are designed to be able to carry loads of concrete casting. This is a great benefit which saves a considerable amount of time, as finishing works can start earlier, and there is no need to buy or lease shores. It is economical as well.

The Ruukki crew was satisfied with the results as they received the evidence that the structure can be implemented into their projects and had another helpful tool to determine the characteristics of the slab.

FIGURES

Figure 1 Visual representation of a composite slab
Figure 2 Different placement of corrugations of the slab
Figure 3 Heat distribution in the slabs with different positions of corrugations
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Figure 7 A trapezoidal cross-section of the slab
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Table 2 Tables of loads
Table 3 Strength of the profiled sheets
Table 4 Deflection of the profiled sheets
Table 5 Strength of inclined sections
Table 6 Bearing stress on the supports
Table 7 Deflection of the slab

CHARTS

Chart 1 Deflection of the slab

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