**MINISTRY OF EDUCATION AND SCIENCE OF UKRAINE**

**O. M. BEKETOV NATIONAL UNIVERSITY**
**of URBAN ECONOMY in KHARKIV**

Metodological guidelines
for execution of design project № 1 on the subject

# “REINFORCED-CONCRETE AND MASONRY STRUCTURES”

**Section 1**

**“Design of monolithic slab and girder floor with beam slabs of a building with a partial framework”**

*(for applicants for higher education in the field of*
*192 – Construction and civil engineering*
*Specialty “Industrial and Civil Building”)*

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N. O. Psurtseva

Reviewer: Doctor of Technical Sciences, Prof. V. S. Shmukler

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**CONTENTS**

## GENERAL GUIDELINES

The most commonly used structures used in construction are floor slabs which are horizontal structures dividing adjacent floors of the house.

According to the design scheme, reinforced flat concrete floor slabs can be divided into two main types: beam and beamless.

With regard to manufacturing method, reinforced concrete flat floor slabs are divided into prefabricated, monolithic and prefabricated-monolithic ones.

In these guidelines, monolithic beam slab and girder floor slabs are considered, where the beams-webs are main and secondary beams.

The main beams rely on the columns and the outer walls of the building. Secondary beams rely on the main beams.

The concrete-filled mesh, which is limited by the main and secondary beams, is a floor slab.

The main beams bays are the distance between the columns or the walls in the chosen direction (6...9 m), the bays of the secondary beams are the distance between the main beams (5...7 m) in the opposite direction. The bay of the slab is a step of secondary beams (1,7...2,5 m).

Thus, the slab in the plane has the dimensions $l_1 \times l_2 = (1{,}7\ldots2{,}5) \times (5\ldots7)$ m. When loaded, such a slab is bent in one (short) direction between the secondary beams; such a slab with the ratio of sides $l_2/l_1 \geq 2$ is called the beam slab. If the step of secondary beams is such that the ratio is $l_2/l_1 < 2$, then the plate under load is bent in two opposite directions. In this case, the slab is called supported along four sides or the contour slab. The calculation of contour slabs is rather complicated and will be considered later on.

# 1 RECOMMENDATIONS FOR THE ARRANGEMENT OF MONOLITHIC SLAB AND GIRDER FLOOR WITH BEAM SLABS. SELECTION OF BAYS AND SIZES OF THE ELEMENTS CROSS-SECTIONS

Monolithic slab and girder floors are used in industrial and civil buildings.

Temporary loads for civil buildings do not usually exceed 6,0 kN/m$^2$; for industrial multi-storey buildings, the temporary load is usually in the range of 6,0...15 kN/m$^2$.

Depending on the size of the floor slab in the plane $L_1 \times L_2$ and the temporary load during the arrangement, it is necessary to choose the direction of the main and secondary beams, arrange the columns and accept the step of secondary beams in such a way, so as to provide recommended bays: for the main beams $l_{m.b.}$ = 6...9 m, for secondary $l_{s.b.}$= 5...7 m, for the slab $l_{sl.}$ = 1,7...2,5 m.

Each of the elements (slab, secondary beam, main beam), according to the design scheme, is a multi-span whole beam, where the extreme supports (on the wall contour) are hinged ones. In this scheme, the bending moments from evenly distributed loads in the extreme bays are larger than in the middle ones, so the extreme bays, when performing arrangement, can be accepted by 15...20 % shorter than the average ones.

The thickness of the monolithic slab is accepted depending on the magnitude of the temporary load and its bay. For civil buildings, the thickness of the slabs is 55...70 mm, while for industrial ones – 70...100 mm.

Recommended cross-sectional dimensions:

– for the main beam:

cross section height (including slab thickness):

$$h_{m.b.} = (1/10\ldots1/12)\ l_{m.b.},$$

cross section width: $b_{m.b.} = (0,3\ldots0,5)h_{m.b.}$.

Finally, the dimensions of the main beam cross section are assumed as multiple of 50 mm;

– for the secondary beam:

cross section height (including slab thickness):

$$h_{s.b.} = (1/15\ldots1/20)l_{s.b.},$$

cross section width: $b_{s.b.} = (0{,}3\ldots0{,}5)h_{s.b.}$.

Finally, the dimensions of the cross section of the secondary beam are assumed as multiple of 50 mm.

## 2 AN EXAMPLE OF CALCULATION FOR ELEMENTS OF MONOLITHIC SLAB AND GIRDER FLOOR WITH BEAM SLABS

The floor slab with dimensions in the plane (in axes) 20 × 30 m are considered.

The building type – industrial. Characteristic value of the temporary load $v_n$ = 8,0 kN/m$^2$. External walls – brick, 510 mm thick.

The composition of the floor slab is assumed as shown in figure 2.1.

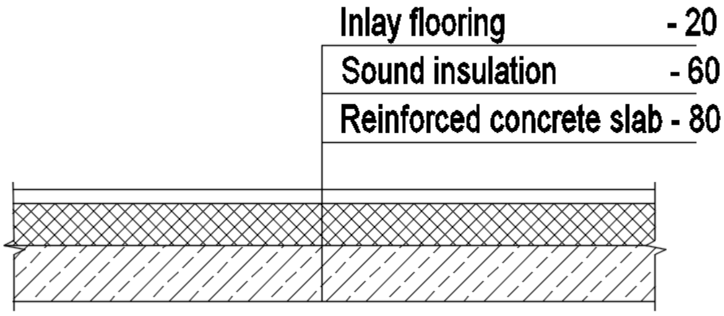


Figure 2.1 – The composition of the floor

For a civil building the floor slab composition with different floor options is assumed in compliance with architectural proposal.

According to the arrangements recommendations, we assume the location of columns with the bays of the main beams, $l_{m.b.1}$ = 6,6 м, $l_{m.b.2}$ = 6,8 m.

All beams of secondary beams $l_{s.b.}$ = 6,0 m. The secondary beams are to be arranged with the step 2,2…2,3 m (fig. 2.2).

The slab thickness is assumed as $h_f$ = 80 mm.

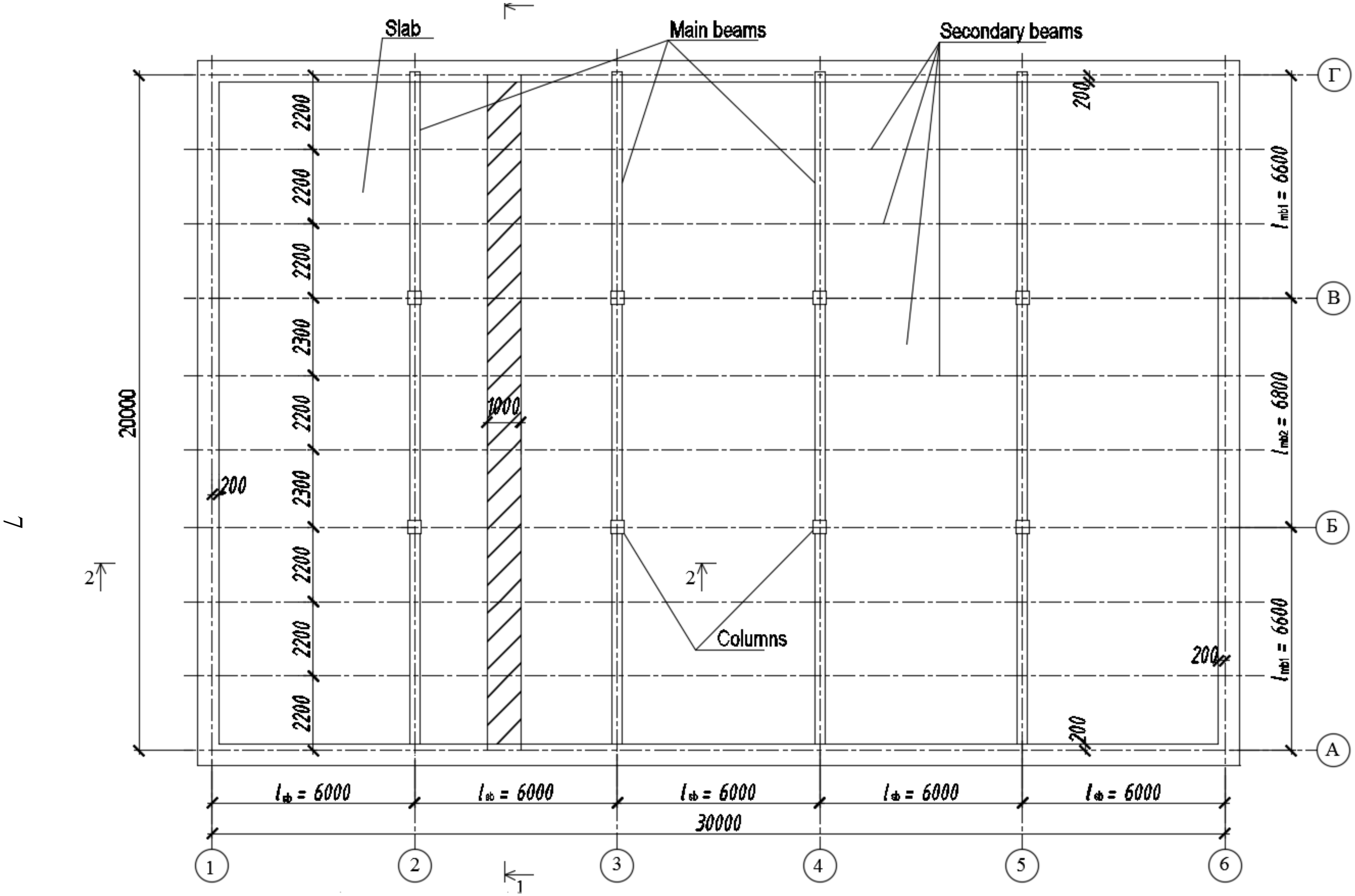


Figure 2.2 – The arrangement of the beams of the floor

## 2.1 The floor slab loading

Floor slabs loads are determined in tabular form (table 2.1).

Table 2.1 – Collecting loads on the overlap

| Load type | Characteristic load value, kN/m$^2$ | Reliability index $\gamma_f$ | Design load value, kN/m$^2$ |
|---|---|---|---|
| A) Constant (***g***) | | | |
| Inlay flooring ($\delta$ = 20 mm, $\rho$ = 20 kN/m$^3$) 0,02·20 | 0,4 | 1,3 | 0,52 |
| Sound insulation – foam concrete ($\delta$=60 mm, $\rho$ = 8 kN/m$^3$) 0,06·8 | 0,48 | 1,3 | 0,62 |
| Reinforced concrete slab ($\delta$=80 mm, $\rho$ = 25 kN/m$^3$) 0,08·25 | 2,0 | 1,1 | 2,2 |
| Total constant | | | $g$ = 3,34 |
| Б) Temporary (***v***) | $V_n$ = 8 | 1,2 | $V$=9,6 |
| Total | | | ***q=g+v=* 12,94~13** |

## 2.2 Design scheme of the slab design bays. Design forces

To calculate the slab, a 1 m wide conventional stripe, parallel to the digital axis, is considered. In the cross section 1-1 this band is given in figure 2.3.

Design bays:

$l_{01}$ = 2 200 – 200 + 60 – 150/2 = 1 985 mm = 1,985 m;

$l_{02} = l_{03}$ = 2 200-150 = 2 050 mm = 2,05 m;

$l_{04}$ = 2 300-150 = 2150 mm = 2,15 m.

Bending bay and moments at support in the considered stripe

$M_1 = -M_B = ql_{01}^2/11 = 13 \cdot 1{,}985^2/11 = 4{,}66$ kNm;

$M_2 = M_3 = -M_C = -M_D = ql_{02}^2/16 = 13 \cdot 2.05^2/16 = 3{,}41$ kNm;

$M_4 = ql_{04}^2/16 = 13 \cdot 2{,}15^2/16 = 3{,}76$ kNm.

The calculation scheme and bending moments in the considered stripe of the slab are shown in figure 2.4.

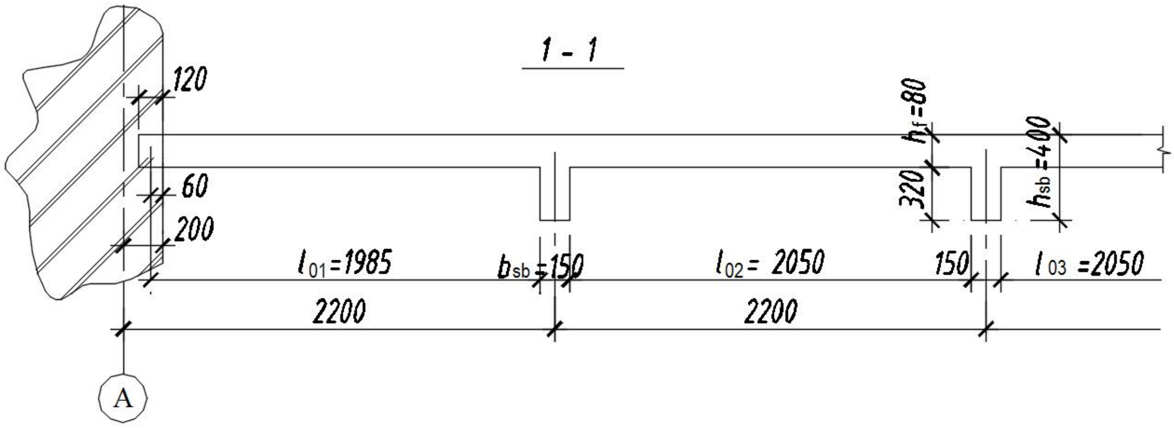


Figure 2.3 – Cross-section of a monolithic slab

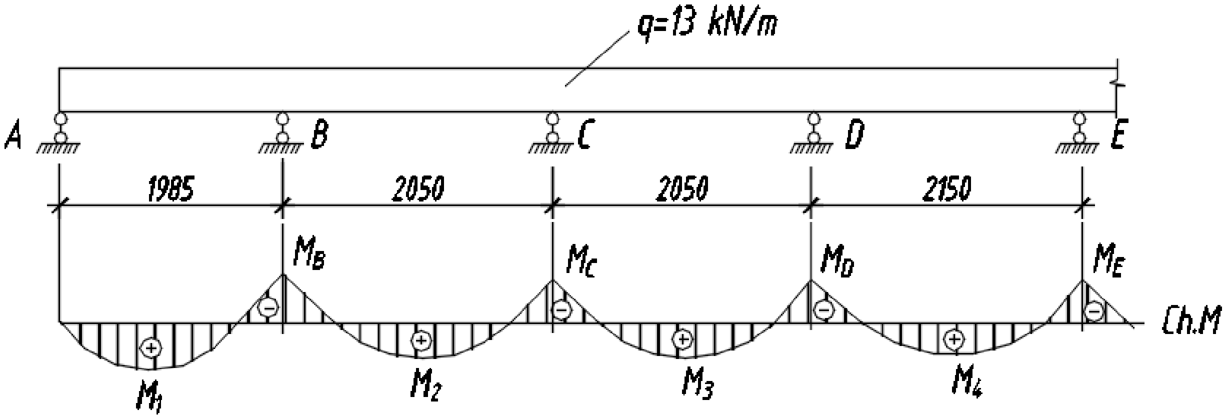


Figure 2.4 – Chart of moments in the slab

## 2.3 Design calculation of the plate. Reinforcement

The slab reinforcement can be carried out in two ways: continuous and separated. The first method is used mainly for civil buildings with a small load on the floor slab. Using this reinforcement method, rolled welded grids are used, which extend in the direction of the main beams. The working longitudinal rebars have diameters not exceeding 5 mm for the B500 grade and a step at least 100 mm.

In the extreme bays of the slab over the main grid, another row of additional grid can be laid to perceive increased bending moments (figure 2.5).

The transverse reinforcement of the grids is constructive, having a minimum diameter of 3...4 mm and a step of 200...250 mm.

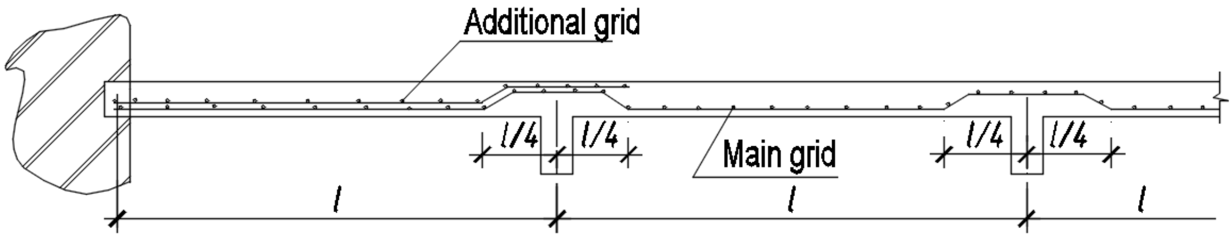


Figure 2.5 – The continuous method of reinforcing the plate

Separated reinforcement with flat grids is used at significant loads in industrial buildings, where the minimum diameter of working rods is 6 mm for A400C grade. The nets are stacked in the bays between the secondary and main beams and above the supports (secondary beams). The working rods are laid in a short direction (transverse). The longitudinal reinforcement of the grids a structural one, it is laid in the longitudinal direction, it has a minimal diameter and a step of 200...250 mm (figure 2.6).

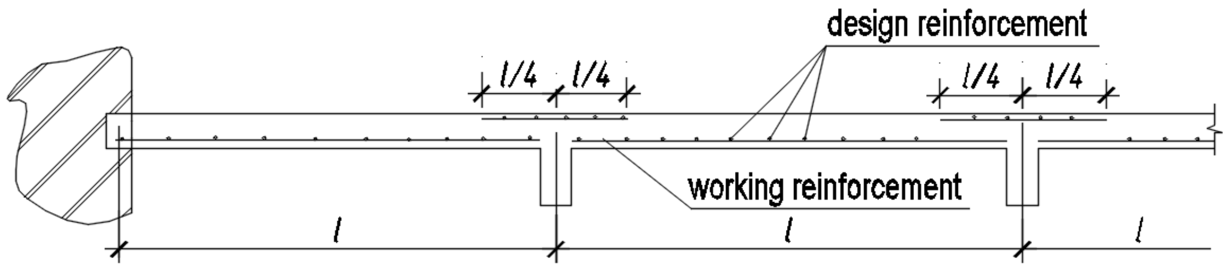


Figure 2.6 – Separate plate reinforcement

In the above example, for an industrial building, it is advisable to take separated reinforcement with flat grids using A400C grade rebars of 6...8 mm in diameter, which has design resistance $f_{yd}$ = 365 MPa = 36,5 kN/cm$^2$ (Appendix A). Concrete class should be assumed as C16/20. Design concrete compressive strength $f_{cd}$ = 11,5 MPa =1,15 kN/cm$^2$ (Appendix C).

In the design calculation the required number of rebars should be determined to ensure the strength of normal cross sections. The strength of sloping cross sections is not checked due to the fairly large width of the calculated stripe of the slab ($b_{sl.}$ = 100 cm).

The design (working) height of the cross section is assumed as d = 5,5 cm.

*a) the first bay and the first intermediate support*

$$\alpha_m = \frac{M_1}{f_{cd} b d^2} = \frac{466}{1{,}15 \cdot 100 \cdot 5{,}5^2} = 0{,}134 < \alpha_R = 0{,}385 \text{ (appendix B)};$$

$$\zeta = 0{,}928 \text{ (Appendix E)};$$

$$A_s = \frac{M_1}{\zeta f_{yd} d} = \frac{466}{0{,}928 \cdot 36{,}5 \cdot 5{,}5} = 2{,}5 \text{ cm}^2.$$

Having assumed a step of working rods as 200 mm, we obtain at the stripe width 1 m 1000/200 = 5 rods. According to the assortment of rebars, we assume Ø8A400C with $A_s$ = 2,51 cm²/m (appendix D). The structural rebars are assumed as Ø3B500 with a step of 250 mm.

The grids grades:

– bay $G1 \frac{3B500-250}{8A400C-200} 5\,700 \times 2\,050$,

– support $G2 \frac{3B500-250}{8A400C-200} 5\,700 \times 1\,100$.

*b) intermediate bay and intermediate supports*

The maximal bending moment is assumed in the assurance factor $M = M_4$ = 3,76 kNm = 376 kNcm:

$$\alpha_m = \frac{376}{1{,}15 \cdot 100 \cdot 5{,}5^2} = 0{,}108; \quad \zeta = 0{,}943;$$

$$A_s = \frac{376}{0{,}943 \cdot 36{,}5 \cdot 5{,}5} = 1{,}97 \text{ cm}^2.$$

Having assumed a step of working rods as 125 mm, we have at the stripe width 1 m 1 000/125 = 8 rods. The necessary area of one rod cross section should be at least 1,97/8 = 0,244 cm². According to the assortment of rebars, we assume Ø6A400C with area of one rod cross section 0,283 cm².

Markets grids:

$$– \text{bay } G3\frac{3B500-250}{6A400C-125}5\,700\times2\,050,$$

$$– \text{support } G4\frac{3B500-250}{6A400C-125}5\,700\times1100.$$

The reinforcement scheme is shown in figure 2.7.

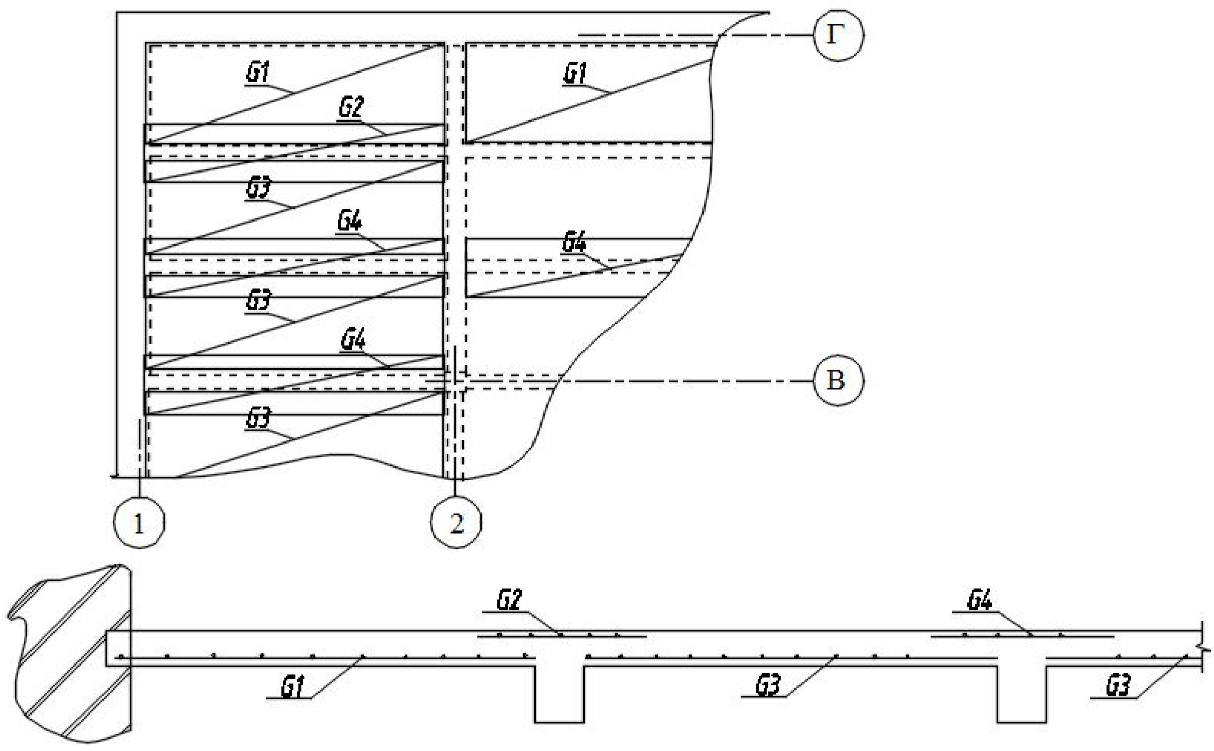


Figure 2.7 – Scheme of separate reinforcement of a monolithic plate

As an example, we consider the option of using rolled grids for the design bending moments.

If possible, we assume the working height of the section as d = 5,5 cm for two-row arrangement of the grids.

The rebars design resistance of B500 grade is $f_{yd}$ = 415 MPa = 41,5 kN/cm$^2$.

The calculation should be started with the intermediate bays:

$$\alpha_m = \frac{376}{1{,}15\cdot100\cdot5{,}5^2} = 0{,}108;\quad \zeta = 0{,}943;$$

$$A_s = \frac{376}{0{,}943\cdot41{,}5\cdot5{,}5} = 1{,}75 \text{ cm}^2.$$

Having assumed a step of longitudinal working rods as 100 mm, we obtain in the stripe 1 m 1 000/100 =10 rods. According the assortment we assume Ø5B500 ($As$ = 1,96 cm$^2$).

In the first span and on the first intermediate support

$$\alpha_m = \frac{466}{1{,}15 \cdot 100 \cdot 5{,}5^2} = 0{,}134; \quad \zeta = 0{,}928;$$

$$A_s = \frac{466}{0{,}928 \cdot 41{,}5 \cdot 5{,}5} = 2{,}2 \text{ cm}^2.$$

Thus, we have a shortage of working rebars in G1 grid in the amount of 2,2 – 1,96 = 0,24 cm$^2$. Therefore, we add an auxiliary grid with a minimum diameter of the working rebars Ø3B500 and with a maximum rods step of 200 mm (the number of rods in the stripe 1000/200 = 5). The cross section area in the additional grid is 0,353 cm$^2$.

Markets grids:

– $G1 \dfrac{5B500-100}{3B500-250} 20\,000 \times B$,

– $G2 \dfrac{3B500-200}{3B500-250} 2\,600 \times B$,

where B – is the width of the grids, which is taken from the process conditions.

The structural concept for the second option is shown in figure 2.8.

## 2.4 Secondary beam. Static calculation

Secondary beam is calculated similarly to the beam slab. The beam perceives the load of its own weight, the weight of the monolithic slab and the floor, as well as the temporary load.

In the cross-section the beam has a T-section with the width of the upper shelf, which is equal to the step of secondary beams.

The design beam bays are assumed subject to the width of the main beams. The first support on the wall is considered to be a hinged one (fig. 2.9).

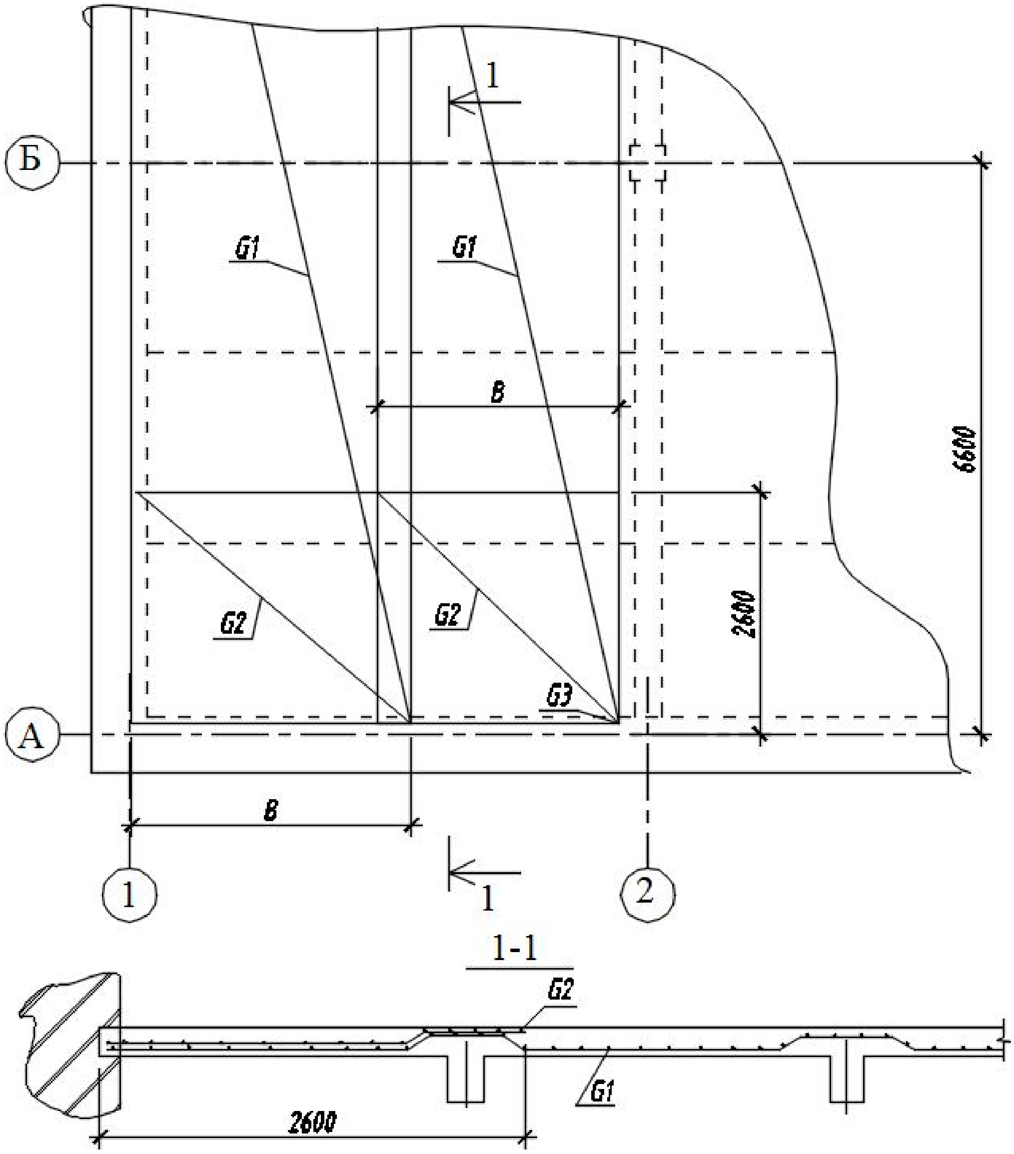

Figure 2.8 – Option for use of roll mesh

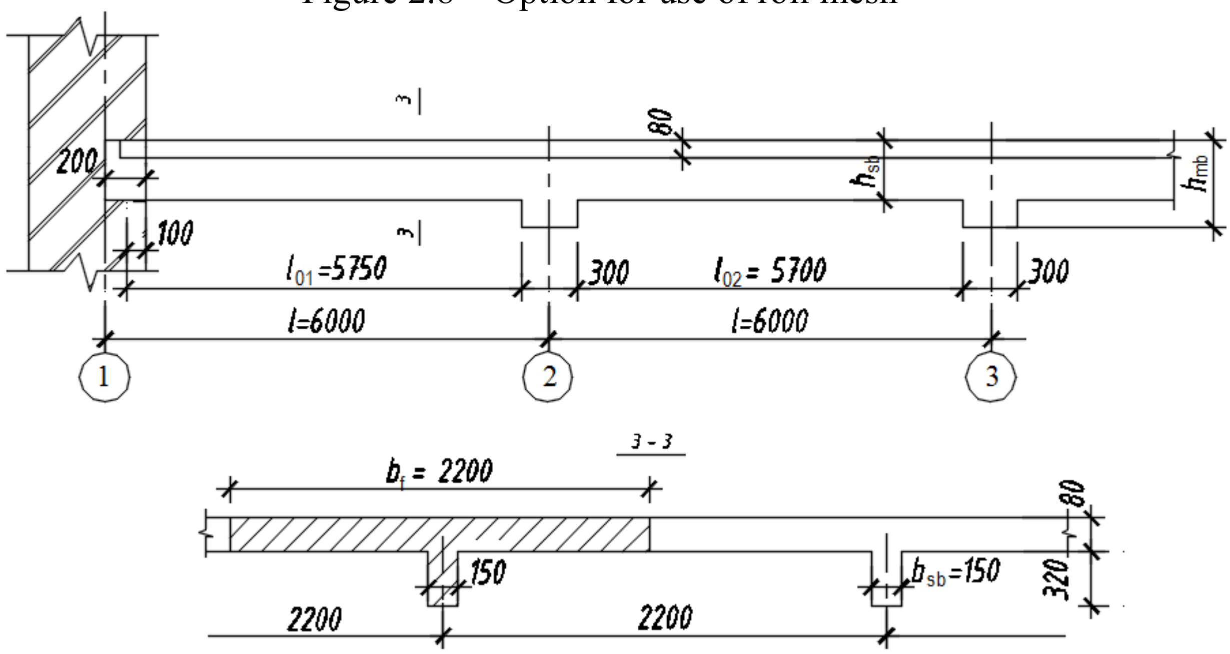

Figure 2.9 – To the definition of the calculated spans of secondary girders

$l_{01} = 6\,000 - 200/2 - 300/2 = 5\,750$ mm = 5,75 m;,

$l_{02} = l_{03} = \ldots = 6\,000 - 300 = 5\,700$ mm = 5,7 m.

*Load*

a) constant

$g = g_{(1\text{m}^2)} \cdot b_f + \gamma_f \cdot$(individual weight of the beam's rib) =

$= 3{,}34 \cdot 2{,}2 + 1{,}1 \cdot 0{,}15 \cdot 0{,}32 \cdot 25 = 8{,}7$ kN/m;

b) temporary

$v = v_{(1\text{m}^2)} \cdot b_f = 9{,}6 \cdot 2{,}2 = 21{,}1$ kN/m;

c) total

$q = g + v = 8{,}7 + 21{,}1 = 29{,}8$ kN/m.

## Design forces

To determine the design forces, consider several possible load schemes. So, in order to obtain the greatest bending moment in the first bay, this bay and all odd bays must have a maximum temporary load with constant loading of all bay (scheme 1). In order to obtain the greatest bending moment in the second bay, this bay and all pair bays must have the maximum temporary load with constant loading of all bays (scheme 2). To determine the minimum bending moment on any support, it is necessary to have the maximum temporary load in two adjacent bays near the considered support with constant loading of all bays (scheme 3).

Epure of moments which combines all the maxima and minima of efforts in all the schemes considered is called the curve of bending moments. Transverse forces according to different schemes are considered similarly to bending moments.

Given the different load patterns (fig. 2.10), the calculated efforts are determined using the formulas:

$$M_1 = ql_{01}^2/11 = 0{,}091ql_{01}^2\ ;\ \ M_B = -\,ql_{01}^2/14 = -\,0{,}0715\ ql_{01}^2;$$

$$M_2 = M_3 = -M_C = -M_D = ql_{02}^2/16 = 0{,}0625\ ql_{02}^2\ ;$$

$$V_A = 0.4ql_{01};\quad V_B^{л} = 0{,}6ql_{01}\ ;\ V_B^{n} = V_C^{л} = V_B^{n} = 0{,}5\ ql_{02}\ .$$

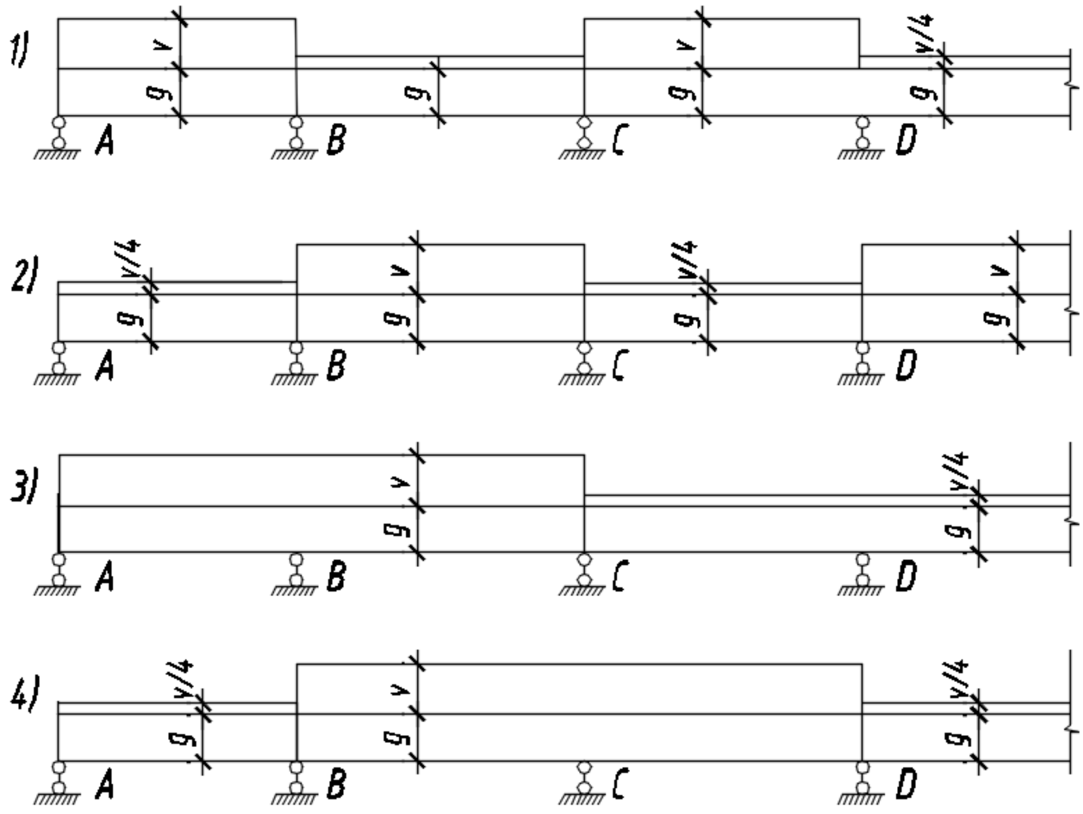

Figure 2.10 - Loading schemes for a secondary beam

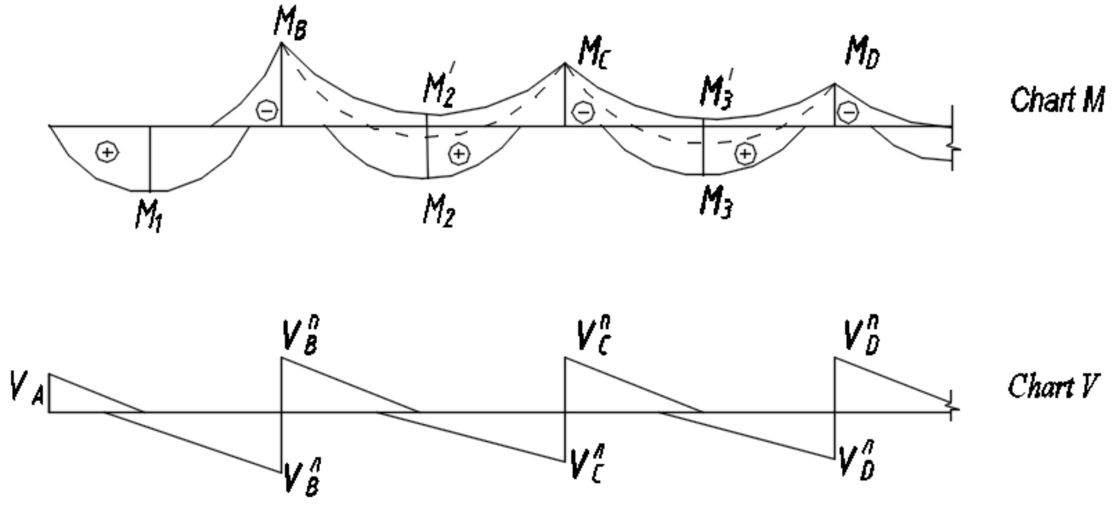


Figure 2.11 – Chart of moments and transverse forces

When constructing curve of bending moments in intermediate bays, negative bending moments may occur. Their values, as well as the zero coordinates of the negative moment in the first bay, are determined by the graph or table, depending on the ratio of the temporary and constant load *v*/*g* values (fig. 2.11, appendix F).

For the example in question

$M_1$ = 0,091·29,8·5,75$^2$ = 89,7 kNm;

-$M_B$ = 0,0715·29,8·5,75$^2$ = 70,4 kNm;

$M_2$ = -$M_C$ = 0,0625·29,8·5,7$^2$ = 60,5 kNm.

At $v/g$ = 21,1/8,7 = 2,42 $M_2'$ = -0,012·29,8·5,7$^2$ = -11,6 kNm.

$V_A$ = 0,4·29,8·5,75= 68,5 kN;

$V_B^l$ = 0,6·29,8·5,75 = 102,8 kN;

$V_B^r$ = $V_C^l$ = $V_B^r$ = 0,5 ·29,8·5,7 =84,9 kN.

## 2.5 Calculation of the secondary beam for the strength in normal sections

The width of the secondary beam shelf does not always coincide with the actual width, which is equal to the step of the secondary beams.

Design or effective shelf width (fig. 2.12)

$b_{eff} = b_w + b_{eff\,1} + b_{eff\,2}$ ,

where $b_w$ – width of the secondary beam rib;

$b_{eff\,1} = (0{,}2b_1 + 0{,}1l_0) \leq 0{,}2l_0 \leq b_1$ ;

$b_{eff\,2} = (0{,}2b_2 + 0{,}1l_0) \leq 0{,}2l_0 \leq b_2$;

$l_0 = 0{,}85l_{01}$ – for extreme bays,

$l_0 = 0{,}7l_{02}$ – for intermediate bays;

$b_1$, $b_2$ – hanging parts of the shelf to the left and to the right of the beam rib (for a constant step of the secondary beams $b_1=b_2=(b_f - b_w)/2$; $b_{eff\,1}=b_{eff\,2}$).

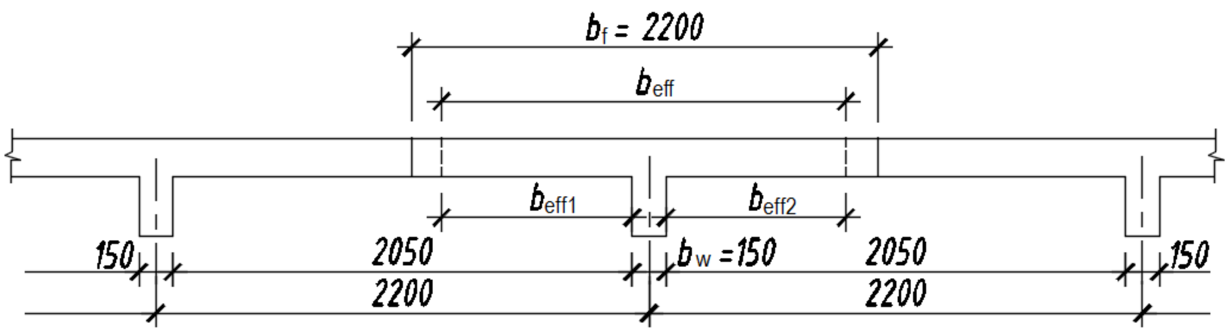


Figure 2.12 – To calculate the effective width of the shelf

In our case

$l_0$ = 0,85 $l_{01}$ = 0,85·5,75 = 4,89 m – for extreme bays;

$b_1 = b_2 = (220\text{-}15)/2 = 102{,}5$ cm;

$l_0 = 0{,}7\ l_{02} = 0{,}7{\cdot}5{,}7 = 3{,}99$ m – for intermediate bays.

The shelf design width

– in the first bays

$$b_{eff\,1} = (0{,}2{\cdot}102{,}5 + 0{,}1{\cdot}489) = 69{,}4 \text{ cm} < 0{,}2{\cdot}489 = 97{,}8 \text{ cm};$$

$$b_{eff\,1} < b_1 = 102{,}5 \text{ cm};$$

$$b_{eff} = b_w + 2b_{eff\,1} = 15 + 2{\cdot}69{,}4 = 154 \text{ cm};$$

– in intermediate bays

$$b_{eff\,1} = (0{,}2{\cdot}102{,}5 + 0{,}1{\cdot}399) = 60{,}4 \text{ cm} < 0{,}2{\cdot}399 = 79{,}8 \text{ cm};$$

$$b_{eff\,1} < b_2 = 102{,}5 \text{ cm};$$

$$b_{eff} = 15 + 2{\cdot}60{,}4 = 136 \text{ cm}.$$

The following is assumed for reinforcement:

– in the bays – rod rebar of A400C grade;

– on the supports – rebar as flat welded grids with rebar of A400C grade.

*The first bay.* The cross section's working height is assumed as $d$ = 36 cm.

Determine the neutral axis location:

$$M_f = f_{cd} b_{eff} h_f (d - 0{,}5h_f) = 1{,}15{\cdot}154{\cdot}8(36 - 4) =$$

$$= 45337{,}6 \text{ kNcm} = 453 \text{ kNm} > M_1 = 89{,}7 \text{ kNm}.$$

The neutral axis is within the shelf, so the cross section is calculated as rectangular:

$$\alpha_m = \frac{M_1}{f_{cd} b_{eff} d^2} = \frac{8970}{1{,}15 \cdot 154 \cdot 36^2} = 0{,}039; \quad \zeta = 0{,}980;$$

$$A_s = \frac{M_1}{\zeta f_{yd} d} = \frac{8970}{0{,}980 \cdot 36{,}5 \cdot 36} = 6{,}96 \text{ cm}^2.$$

We assume 2Ø22A400C ($A_s$ = 7,6 cm$^2$).

*The intermediate bay.*

$M_f$ = 1,15·136·8(36- 4) = 40038 kNcm = 401 kNm > $M_2$ = 60,5 kNm.

The neutral axis is located within the shelf.

$$\alpha_m = \frac{6050}{1{,}15 \cdot 136 \cdot 36^2} = 0{,}030; \quad \zeta = 0{,}984;$$

$$A_s = \frac{6050}{0{,}984 \cdot 36{,}5 \cdot 36} = 4{,}68 \text{ cm}^2.$$

Assume 2Ø18A400C ($A_s$ = 5,09 cm$^2$).

*Support B.*

The cross section's working height is assumed as $d$ = 38 cm. The compressed area is in the beam rib, so the cross section should be calculated as rectangular with the rib's width ($b_w$ =15 cm):

$$\alpha_m = \frac{7040}{1{,}15 \cdot 15 \cdot 38^2} = 0{,}283; \quad \zeta = 0{,}830;$$

$$A_s = \frac{7040}{0{,}830 \cdot 36{,}5 \cdot 38} = 6{,}11 \text{ cm}^2.$$

For the working grid the rods step is assumed as 100 mm. 2200/100 = 22 rods will be located on the shelf actual width. The cross-sectional area of one rod 6,11/22 = 0,278 cm$^2$. Ø6A400C з $A_s$ = 0,283 cm$^2$ can be assumed for reinforcement. In which case, the grid grade

$$G5\frac{3B500-250}{6A400C-100}4\,000\times19\,800.$$

*Support C.*

$$\alpha_m = \frac{6050}{1{,}15 \cdot 15 \cdot 38^2} = 0{,}243; \quad \zeta = 0{,}858;$$

$$A_s = \frac{6050}{0{,}858 \cdot 36{,}5 \cdot 38} = 5{,}08 \text{ cm}^2.$$

We assume the rods step as 200 mm. At the width of the shelf we obtain 2200/200 = 11 rods. The cross-sectional area of one rod is 5,08/11 = 0,462 cm$^2$. We assume Ø8A400S.

$$G6\frac{3B500-250}{8A400C-200}4\,000\times19\,800.$$

Grids G5, G6 are located above the main beams. The grids width is determined as per the recommendations, so that 1/3 of the secondary beam bay would be overlapped bt the grid in each side from the main beam axis.

To save the rebars, one can use the two-level grids arrangement option (fig. 2.13).

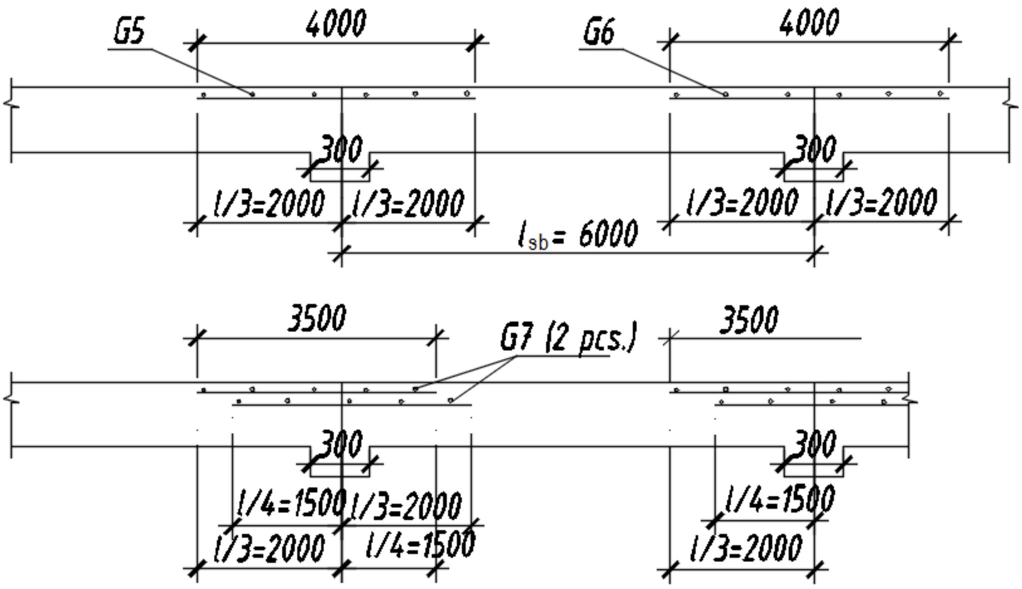


Figure 2.13 – Options for placing nets above the main beams

## 2.6 Calculation of the secondary beam for the strength in sloping sections

In most cases, the calculation of the sloping cross sections strength is performed for the largest value of transverse force. In the example under consideration, the maximum transverse force $V_{max} = V_B^л = 102{,}8$ kN.

At the first stage, the number of transverse rods in the cross section and their diameter are assumed, depending on the longitudinal reinforcement.

We assume transverse reinforcement (collar clamps) 2Ø8A240C ($A_{sw} = 1{,}01$ cm$^2$).

The collar clamps step is $S_w \le 0{,}75d = 27$ cm. We assume $S_w = 20$ cm.

The *concrete carrying capacity* should be determined according to the larger of the two formulas:

$$V_{Rd,c\,1} = (C_{Rd,c} K \sqrt[3]{100\,\rho_1 f_{ck}})\, b_w d\ ,$$

$$V_{Rd,c\,2} = (0{,}035\sqrt{f_{ck}K^3})b_w d\ ,$$

where $C_{Rd,c} = 0{,}18/\gamma_c = 0{,}18/1{,}3 = 0{,}1385$;

$\rho_1 = A_s/b_w d = 7{,}6/15\cdot 36 = 0{,}014$;

$K = 1+\sqrt{\dfrac{200}{d}} = 1+\sqrt{\dfrac{200}{360}} = 1{,}745 < 2.$

$V_{Rd,c\,1}$ =$(0{,}1385\cdot 1{,}745\sqrt[3]{100\cdot 0{,}014\cdot 15})150\cdot 360 = 36060$ N = 36,06 kN;

$V_{Rd,c\,2} = (0{,}035\sqrt{15\cdot 1{,}745^3})150\cdot 360 = 8133{,}7$ N = 8,13 kN.

We assume the large value $V_{Rd,c}$ = 36,06 kN < $V_{Ed}$ = 102,8 kN.

Crosswise reinforcement is required based on calculation.

Collar clamps carrying capacity should be determined using the formula:

$$V_{Rd,s} = A_{sw} z f_{ywd} ctg\theta/s\ ,$$

where $z = 0{,}9d = 0{,}9\cdot 36 = 32{,}4$ cm; $f_{ywd} = 175$ MPa;

$Ctg\theta$ should be determined subject to the value $\dfrac{V_{Ed}}{b_w d} = \dfrac{102800(н)}{150\cdot 360}$ = 1,904 as per the graph (fig. 2.14).

Ctgθ = 2,5;

$$V_{Rd,s} = 1{,}01\cdot 32{,}4\cdot 17{,}5\cdot 2{,}5\ /20 = 71{,}58 \text{ kN}$$

Total load carrying capacity of the sloping section

$$V_{Rd} = V_{Rd,c} + V_{Rd,s} = 36{,}06 + 71{,}58 = 107{,}64 \text{ kN} > V_{Ed} = 102{,}8 \text{ kN}.$$

The carrying capacity of the sloping section is sufficient.

If the latter condition is not fulfilled, the clamps diameter shall be increased, as well as the class of reinforcement or the step of clamps shall be reduced.

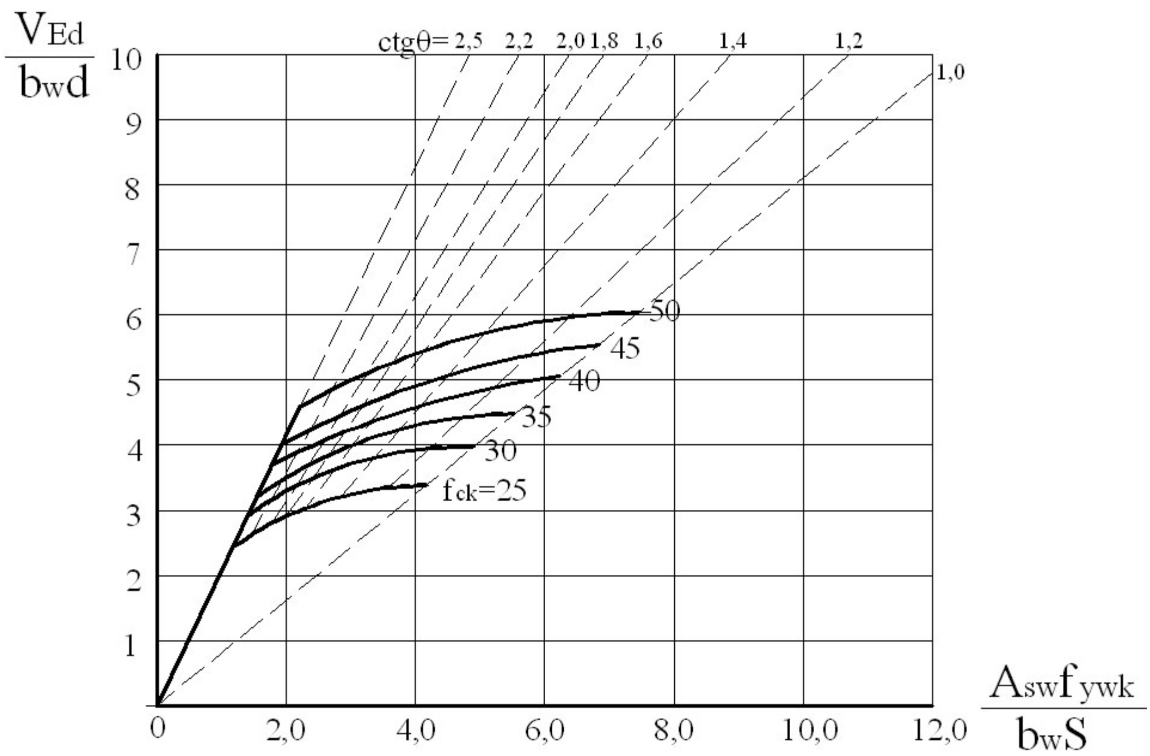


Figure 2.14 – Schedule for determining

## 2.7 Construction of the secondary beam

The reinforcement of the secondary beam is performed via welded frameworks and grids.

Welded frameworks in each bay are formed from the working lower reinforcement, as defined in the preliminary calculation, the upper structural reinforcement and transverse reinforcement (clamps).

The upper reinforcement is assumed to be 1,5 ... 2 times less in diameter than the working reinforcement and may be lower in the grade. The amount of this reinforcement is usually sufficient to accommodate the negative bending moments $M_2'$.

The frameworks are brought to the supports in the area of the main beams and interconnected with each other by the dowel bars, which are passed through the main beams.

The collar clamps are placed in the frameworks according to the calculation.

Grids of secondary beams are located above the frameworks in one or two rows.

The general view of the reinforcement of the secondary beam is shown in figure 2.15.

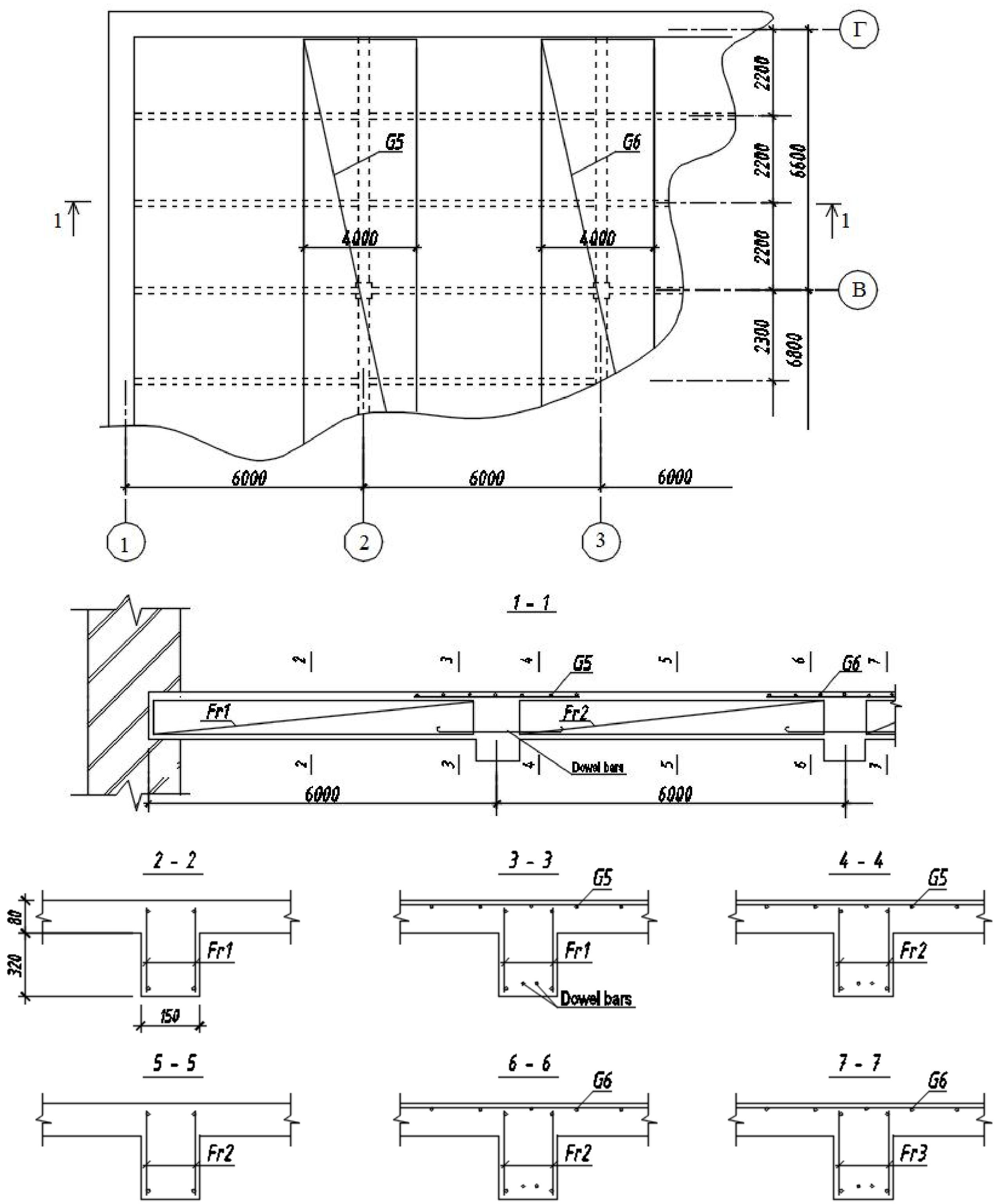


Figure 2.15 – Reinforcement of the secondary beam (start)

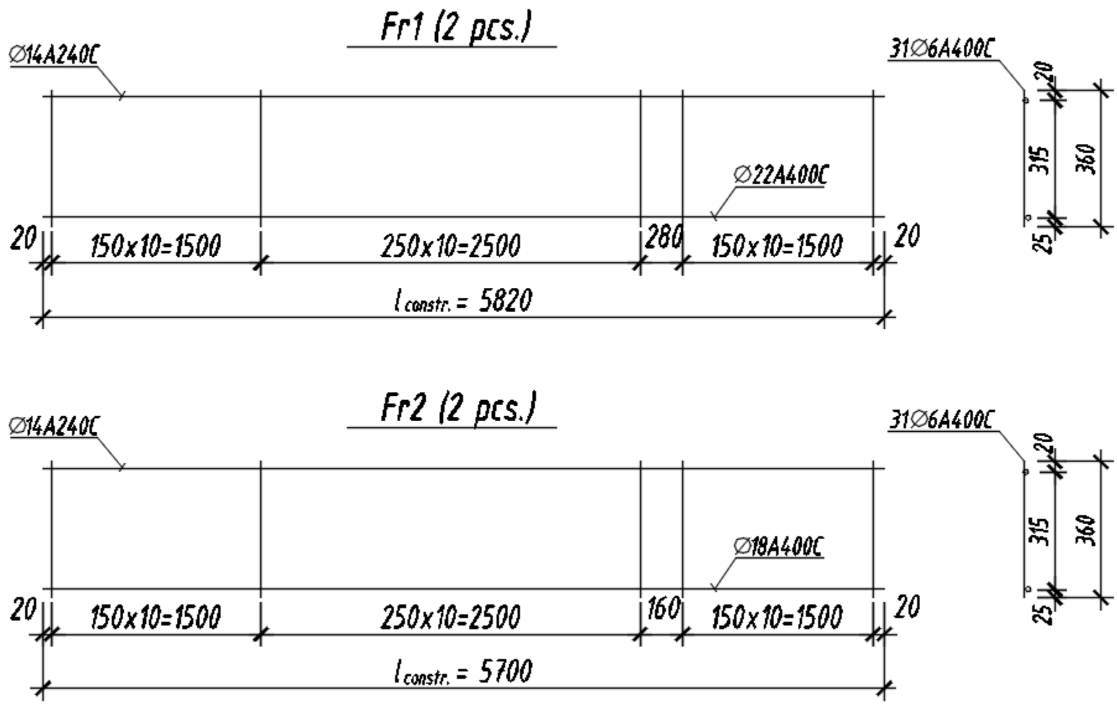


Figure 2.15 – Reinforcement of the secondary beam (continuation)

## 3 PRINCIPLES OF THE MAIN BEAM CALCULATION

The load on the main beam is considered as concentrated forces, applied in the areas of the secondary beams support. The proper weight of the main beams can be taken into account as a uniformly distributed load, or to be normalized to the concentrated forces, which are also applied in the places of the secondary beams support.

The calculated forces M and V are also obtained from the enveloping scenes when considering the different application schemes of loads. In most cases, after the construction of the curve of bending moments, redistribution of forces from the supports in the bay is made.

The reinforcement of the main beams is performed by welded flat or spatial bay and supporting frameworks.

In some cases reinforcement can be performed by separate rods. In these cases, to accommodate the transverse forces, turnovers are installed in addition to the collar clamps.

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## APPENDIX A

## Estimated concrete supports with axial compression and tension, modulus of elasticity

| Class of concrete for compressive strength | Estimated concrete resistance when calculating for I group of boundary states, MPa | | Initial modulus of elasticity in compression $E_{cm}\cdot 10^3$. MPa | Note |
|---|---|---|---|---|
| | with compression $f_{cd}$ | when stretching $f_{ctd}$ | | |
| C8/10 | 6,0 | 0,53 | 18,0 | The value of the elastic modulus is given for heavy concrete |
| C12/15 | 8,5 | 0,73 | 23,0 | |
| C16/20 | 11,5 | 0,87 | 27,0 | |
| C20/25 | 14,5 | 1,0 | 30,0 | |
| C25/30 | 17,0 | 1,2 | 32,5 | |
| C30/35 | 19,5 | 1,33 | 34,5 | |
| C32/40 | 22,0 | 1,4 | 36,0 | |

## APPENDIX B

## The value of the boundary factor $\alpha_R$

| Armature class | Class of heavy concrete | | |
|---|---|---|---|
| | C12/15 | C16/20 | C20/25 |
| A240C | 0,423 | 0,420 | 0,418 |
| A400C | 0,387 | 0,385 | 0,381 |
| A500C | 0,370 | 0,367 | 0,363 |
| B500 | 0,361 | 0,358 | 0,354 |

## APPENDIX C

## Calculation relief valves. Modulus of elasticity

| Armature class | Estimated resistance of the armature when calculating for the I group of boundary states, MPa | | | Modulus of elasticity $E_s$*$10^4$, MPa |
|---|---|---|---|---|
| | when stretching | | with compression $f_{yd}'$ | |
| | in the longitudinal direction $f_{yd}$ | in the transverse direction when calculating sloping sections $f_{ywd}$ | | |
| A240C | 225 | 170 | 225 | 21 |
| A400C | 365 | 285 | 365 | 21 |
| A500C<br>Ø8…22<br>Ø25…40 | <br>435<br>415 | <br>300<br>300 | <br>435<br>415 | 21 |
| B500 | 415 | 300 | 375 | 19 |

APPENDIX D

**Sorting of reinforcing steel according to State Standard 3760: 2006**

| Diameter mm | Calculated cross-sectional area, cm2, with the number of rods | | | | | | | | | Theoretical weight kg | Diameters for valves of classes | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | | A240C | A400C | B500 | Bp1200 - Bp1500 |
| 3 | 0,071 | 0,141 | 0,212 | 0,283 | 0,353 | 0,424 | 0,495 | 0,565 | 0,636 | 0,055 | | | + | + |
| 4 | 0,126 | 0,251 | 0,377 | 0,502 | 0,628 | 0,754 | 0,879 | 1,005 | 1,130 | 0,099 | | | + | + |
| 5 | 0,196 | 0,393 | 0,589 | 0,785 | 0,982 | 1,178 | 1,375 | 1,571 | 1,767 | 0,154 | | | + | + |
| 5,5 | 0,238 | 0,48 | 0,71 | 0,95 | 1,19 | 1,43 | 1,67 | 1,90 | 2,14 | 0,187 | + | | | |
| 6 | 0,283 | 0,57 | 0,85 | 1,13 | 1,41 | 1,7 | 1,98 | 2,26 | 2,54 | 0,222 | + | + | | + |
| 7 | 0,385 | 0,77 | 1,15 | 1,54 | 1,92 | 2,31 | 2,69 | 3,08 | 3,46 | 0,302 | | | | + |
| 8 | 0,503 | 1,01 | 1,51 | 2,01 | 2,51 | 3,02 | 3,52 | 4,02 | 4,53 | 0,395 | + | + | | + |
| 10 | 0,785 | 1,57 | 2,36 | 3,14 | 3,93 | 4,71 | 5,5 | 6,28 | 7,07 | 0,617 | + | + | | |
| 12 | 1,131 | 2,26 | 3,39 | 4,52 | 5,65 | 6,79 | 7,92 | 9,05 | 10,18 | 0,888 | + | + | | |
| 14 | 1,539 | 3,08 | 4,62 | 6,16 | 7,69 | 9,23 | 10,77 | 12,31 | 13,85 | 1,208 | + | + | | |
| 16 | 2,011 | 4,02 | 6,03 | 8,04 | 10,05 | 12,06 | 14,07 | 16,08 | 18,10 | 1,578 | + | + | | |
| 18 | 2,545 | 5,09 | 7,63 | 10,18 | 12,72 | 15,27 | 17,81 | 20,36 | 22,90 | 1,998 | + | + | | |
| 20 | 3,142 | 6,28 | 9,42 | 12,56 | 15,71 | 18,85 | 21,99 | 25,13 | 28,27 | 2,466 | + | + | | |
| 22 | 3,801 | 7,60 | 11,40 | 15,20 | 19,00 | 22,81 | 26,61 | 30,41 | 34,21 | 2,984 | + | + | | |
| 25 | 4,909 | 9,82 | 14,73 | 19,63 | 25,54 | 29,45 | 34,36 | 39,27 | 44,18 | 3,84 | + | + | | |
| 28 | 6,158 | 12,32 | 18,47 | 24,63 | 30,79 | 36,95 | 43,10 | 49,26 | 55,42 | 4,83 | + | + | | |
| 32 | 8,043 | 16,09 | 24,13 | 32,17 | 40,21 | 48,26 | 56,30 | 64,34 | 72,38 | 6,31 | + | + | | |
| 36 | 10,179 | 20,36 | 30,54 | 40,72 | 50,89 | 61,07 | 71,25 | 81,43 | 91,61 | 7,99 | + | + | | |
| 40 | 12,566 | 25,13 | 37,7 | 50,27 | 62,83 | 75,40 | 87,96 | 100,53 | 113,10 | 9,865 | + | + | | |

APPENDIX E

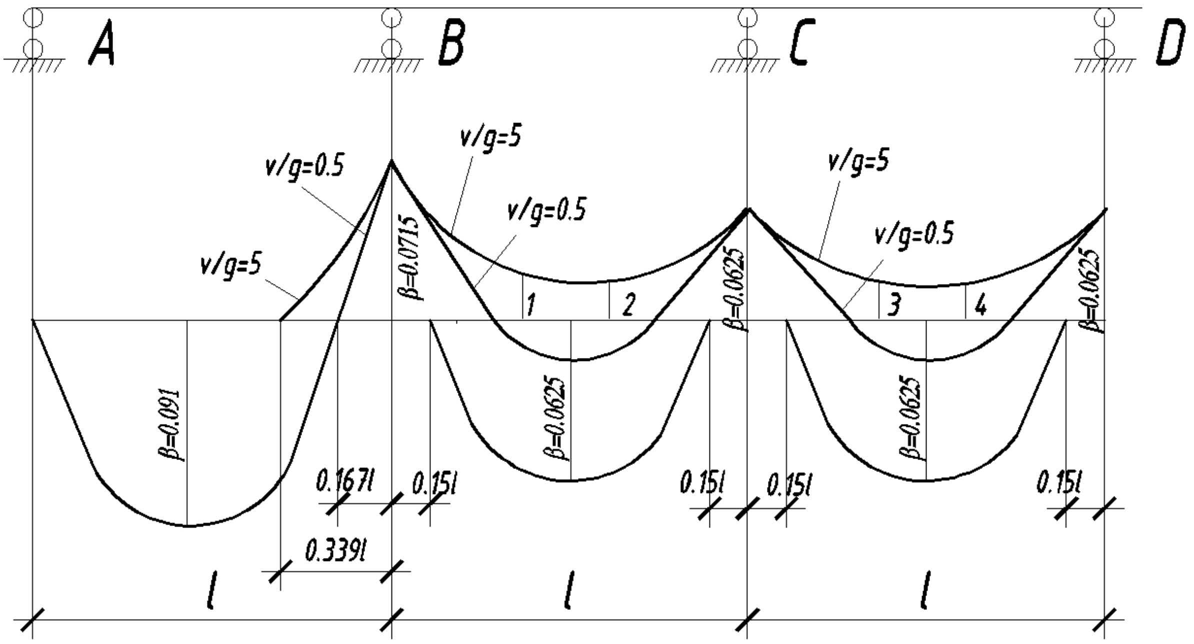
**Value of coefficients $\alpha_m$, $\xi$ ma $\zeta$**

| $\xi$ | $\zeta$ | $\alpha_m$ | $\xi$ | $\zeta$ | $\alpha_m$ | $\xi$ | $\zeta$ | $\alpha_m$ |
|---|---|---|---|---|---|---|---|---|
| 0,01 | 0,996 | 0,008 | 0,26 | 0,896 | 0,186 | 0,51 | 0,796 | 0,325 |
| 0,02 | 0,992 | 0,016 | 0,27 | 0,892 | 0,193 | 0,52 | 0,792 | 0,329 |
| 0,03 | 0,988 | 0,024 | 0,28 | 0,888 | 0,199 | 0,53 | 0,788 | 0,334 |
| 0,04 | 0,984 | 0,031 | 0,29 | 0,884 | 0,205 | 0,54 | 0,784 | 0,339 |
| 0,05 | 0,980 | 0,039 | 0,3 | 0,880 | 0,211 | 0,55 | 0,780 | 0,343 |
| 0,06 | 0,976 | 0,047 | 0,31 | 0,876 | 0,217 | 0,56 | 0,776 | 0,348 |
| 0,07 | 0,972 | 0,054 | 0,32 | 0,872 | 0,223 | 0,57 | 0,772 | 0,352 |
| 0,08 | 0,968 | 0,062 | 0,33 | 0,868 | 0,229 | 0,58 | 0,768 | 0,356 |
| 0,09 | 0,964 | 0,069 | 0,34 | 0,864 | 0,235 | 0,59 | 0,764 | 0,361 |
| 0,1 | 0,960 | 0,077 | 0,35 | 0,860 | 0,241 | 0,6 | 0,760 | 0,365 |
| 0,11 | 0,956 | 0,084 | 0,36 | 0,856 | 0,247 | 0,62 | 0,752 | 0,373 |
| 0,12 | 0,952 | 0,091 | 0,37 | 0,852 | 0,252 | 0,64 | 0,744 | 0,381 |
| 0,13 | 0,948 | 0,099 | 0,38 | 0,848 | 0,258 | 0,66 | 0,736 | 0,389 |
| 0,14 | 0,944 | 0,106 | 0,39 | 0,844 | 0,263 | 0,68 | 0,728 | 0,396 |
| 0,15 | 0,940 | 0,113 | 0,4 | 0,840 | 0,269 | 0,7 | 0,720 | 0,403 |
| 0,16 | 0,936 | 0,120 | 0,41 | 0,836 | 0,274 | 0,72 | 0,712 | 0,410 |
| 0,17 | 0,932 | 0,127 | 0,42 | 0,832 | 0,280 | 0,74 | 0,704 | 0,417 |
| 0,18 | 0,928 | 0,134 | 0,43 | 0,828 | 0,285 | 0,76 | 0,696 | 0,423 |
| 0,19 | 0,924 | 0,140 | 0,44 | 0,824 | 0,290 | 0,78 | 0,688 | 0,429 |
| 0,20 | 0,920 | 0,147 | 0,45 | 0,820 | 0,295 | 0,8 | 0,680 | 0,435 |
| 0,21 | 0,916 | 0,154 | 0,46 | 0,816 | 0,300 | 0,85 | 0,660 | 0,449 |
| 0,22 | 0,912 | 0,161 | 0,47 | 0,812 | 0,305 | 0,9 | 0,640 | 0,461 |
| 0,23 | 0,908 | 0,167 | 0,48 | 0,808 | 0,310 | 0,95 | 0,620 | 0,471 |
| 0,24 | 0,904 | 0,174 | 0,49 | 0,804 | 0,315 | 1 | 0,600 | 0,480 |
| 0,25 | 0,900 | 0,180 | 0,50 | 0,800 | 0,320 | – | – | – |

$$\alpha_m = 0{,}8\xi(1 - 0{,}4\xi);\ \zeta = (1 – 0{,}4\xi)$$

APPENDIX F

## Study courses of calculation moments $M = \beta(g + v)l^2$




| $v/g$ | Coefficient β for points | | | |
|---|---|---|---|---|
| | 1 | 2 | 3 | 4 |
| 0,5 | 0,022 | 0,024 | 0,028 | 0,028 |
| 1,0 | 0,016 | 0,009 | 0,013 | 0,013 |
| 1,5 | -0,003 | 0 | 0,004 | 0,004 |
| 2,0 | -0,009 | -0,006 | -0,003 | -0,003 |
| 2,5 | -0,012 | -0,009 | -0,006 | -0,006 |
| 3,0 | -0,016 | -0,014 | -0,01 | -0,01 |
| 3,5 | -0,019 | -0,017 | -0,013 | -0,013 |
| 4,0 | -0,021 | -0,021 | -0,015 | -0,015 |
| 4,5 | -0,022 | -0,02 | -0,016 | -0,016 |
| 5,0 | -0,024 | -0,021 | -0,018 | -0,018 |

*Виробничо-практичне видання*

Методичні рекомендації
до виконання курсового проєкту № 1,
практичних занять та самостійної роботи
з навчальної дисципліни

# «ЗАЛІЗОБЕТОННІ КОНСТРУКЦІЇ»

**Розділ 1**
**Проєктування монолітного залізобетонного ребристого перекриття з балковими плитами для будівлі з неповним каркасом»**

*(для здобувачів вищої освіти
спеціальності 192 – Будівництво та цивільна інженерія
освітньої програми “Промислове та цивільне будівництво”)*

(англ. мовою)

Укладачі: **РЕЗНІК** Петро Аркадійович,
**ПСУРЦЕВА** Ніна Олексіївна

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