

National Technical University  
“Dnipro University of Technology”  
Ministry of Science and Education of Ukraine

Faculty of Construction

Department of Construction, Geotechnics and Geomechanics

**Reinforced Concrete and Stone Structures**

Tutorial on term project

Dnipro  
2019

Reinforced Concrete and Stone Structures. Tutorial on term project. For students of specialty 192 "Construction and civil engineering"/ VolkovaV.E. – D: National Technical University, 2020-40 p.

ПРОЕКТ

Ministry of Science and Education of Ukraine  
National Technical University “Dnipro University of Technology”  
Department of Construction, Geotechnics and Geomechanics  
**A S S I G M E N T**  
for the term project on reinforced concrete and stone structures

**1. Description of the project**

It is necessary to calculate and execute construction of elements of reinforced concrete structures of an inter-storey monolithic floor of a multi-storey building with an incomplete frame and brick walls.

**2. Project scope**

2.1. Perform the layout of reinforced concrete monolithic flooring.

2.2. Perform calculations and design: monolithic plate; monolithic secondary beams; monolithic head beam; monolithic columns of the first floor; - monolithic foundation.

2.3. Perform calculations and construction: ribbed reinforced concrete plate; reinforced concrete crossbar; brick floor of the first floor.

2.4. The drawing is done in 2 sheets: A1-1 sheet format; A2-1 sheet format.

2.5. Explanatory note to the project is drawn up on sheets of A4 size and should have: calculation diagrams, sketches, calculations, brief comments and references to the used literature. Student can use a computer for calculation and drawings of the course project.

**3. Selection of initial data**

Necessary data of the tasks are programmed according to the code, which consists of the student surname letters.

**Table 0.1**

Code	Letters	Unit	INITIAL DATE														
			1	2	3	4	5	АЛ Х	БІ М	ВЕ Ч	ГО Ш	ДУ Ш	РИ Н	ЖЄ С	ТЮ З	ПЯ Ц	КФ Ь
1	Building length, $L$ Number of floors	м	32	34	36	32	30	34	36	36	32	30					
			5	4	5	6	3	4	5	3	5	6					
2	Building width, $B$ Window width	м	18	20	22	25	18	24	26	22	21	19					
			3,6	3,0	3,2	3,6	3,4	3,0	3,2	3,4	3,0	3,6					
	Flooring	кНМ <sub>2</sub>	1,2	1,4	1,1	1,0	1,3	1,2	1,1	1,4	1,0	1,3					
3	Live load $V$ Reinforcement steel class	кНМ <sub>2</sub>	7.0	10	8.0	12.0	11.0	9.0	11.0	13.0	8.0	12.0					
			A300	A400	A300	A400	A400	A300	A400	A400	A300	A400					
4	Heavy-weight concrete class Floor height $H$	м	C15	C20	C25	C20	C15	C25	C20	C15	C20	C25					
			3,6	4,2	4,8	5,4	3,6	4,2	4,8	5,4	4,2	6,0					
5	Load safety factor Design strength	$\gamma_f$ soil Ro, MPa	1,3	1,25	1,2	1,3	1,25	1,2	1,3	1,25	1,3	1,3					
			0,26	0,19	0,20	0,31	0,30	0,25	0,19	0,20	0,27	0,19					

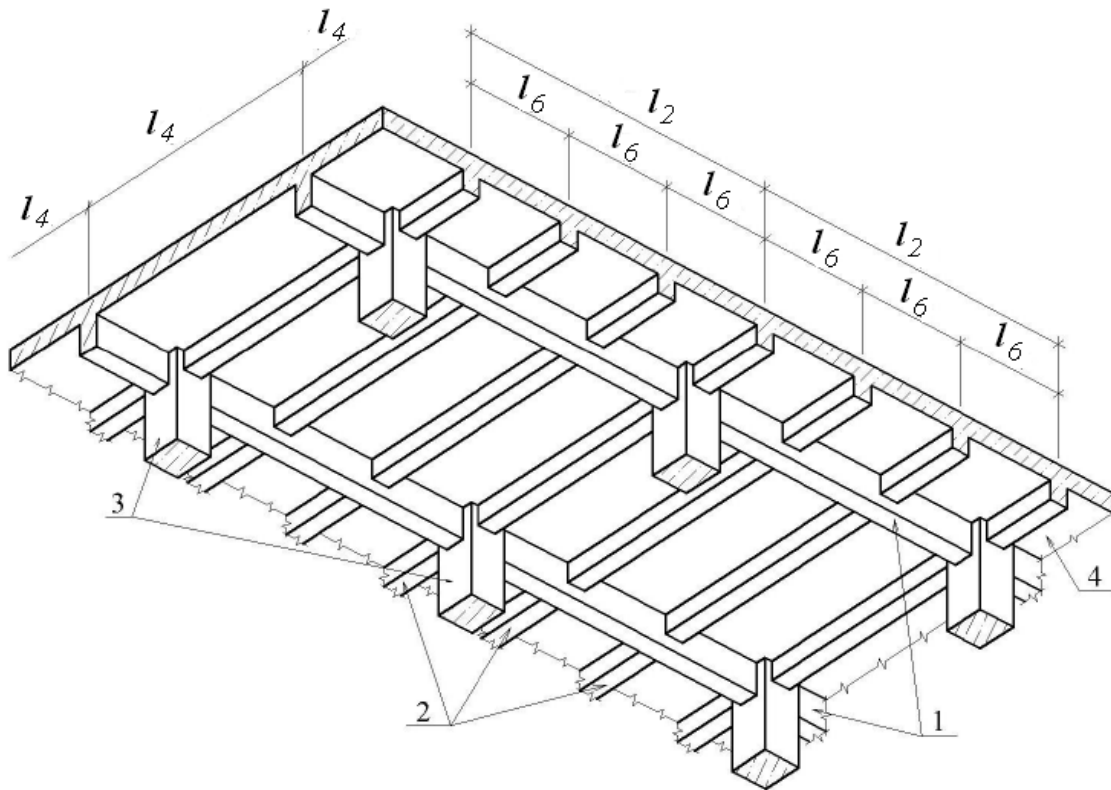


Figure 1. Interfaced ribbed floor: 1-head beam; 2-secondary beam; 3- column; 4- plate.

Table 0.2

Length of secondary beams span, m

Length of building, $L$	$l_3$	$l_4$
30	6	6
31	5	5,25
32	5	5,5
34	5	6
36	6	6

Table 0.3

Length of head beams and plates span, m

Building width, $B$	Spans of head beams		Spans of plates	
	$l_1$	$l_2$	$l_5$	$l_6$
18	6	6	2	2
19	6,2	6,6	1,8	2,2
20	6,55	6,9	1,95	2,3
22	7,25	7,5	2,25	2,5
<b>24</b>	<b>6</b>	<b>6</b>	<b>2</b>	<b>2</b>
25	6,2	6,3	2	2,1
26	6,4	6,6	2	2,2

## 1. PRELIMINARY DETERMINATION SIZES OF PLATE AND CROSS-SECTION BEAMS

For preliminary determination of thickness of a plate we can use formula

$$h'_f = (l_6 - 0.2)\sqrt{(l_6 - 0.2) + v},$$

The thickness of the plates of industrial buildings is recommended to be taken in advance depending on the time load and their span. The minimum thickness of the plate of monolithic overlap is taken 6 cm from the condition of technology of concreting plate structures. The preliminary value of the thickness of the plate is assigned in accordance with table. 2, taking into account the design experience and taken with a multiplicity of 1 cm

The preliminary dimensions of the cross-section of the beams, taking into account their weight, based on the design experience, we accept depending on the span:

$$\text{- head beams, } h_1 = \left(\frac{1}{7} \div \frac{1}{15}\right) \cdot l_2; \quad \text{- secondary beams}$$

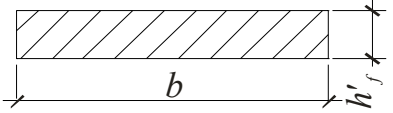
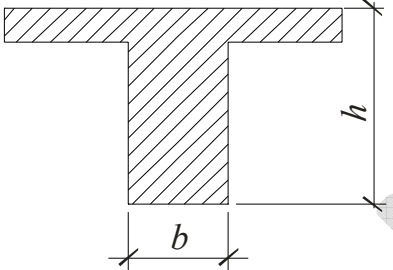
$$h_2 = \left(\frac{1}{12} \div \frac{1}{20}\right) \cdot l_4,$$

$$\text{- width of beam } b = \left(\frac{1}{2} \div \frac{1}{3}\right) \cdot h.$$

The cross-sectional dimensions of the beams should be taken with a multiplicity of 5 cm at cm and with a multiplicity of 10 cm at cm.

**Table 1**

Previous cross-sectional dimensions of the overlapping elements

Element name	View	Element height, cm	Element width, cm
Plate		$h'_f=9$	$b=100$
secondary beam		$h_2 = \left( \frac{1}{12} \div \frac{1}{20} \right) \cdot l_4$ = = $600/12=50$	$b_2 = \left( \frac{1}{2,5} \right) \cdot h$ = = $50/2,5=20$
head beams		$h_1 = \left( \frac{1}{7} \div \frac{1}{15} \right) \cdot l_2 =$ = $720/10=72$	$b_1 = \left( \frac{1}{2,5} \right) \cdot h =$ = $72/2,5=28,8$

Accepted cross-sectional dimensions of the overlapping elements are given in Table. 3. Considering the recommendations, we accept the following cross-sectional dimensions of the elements.

Table 2

Recommended beam section sizes:

beam height $h$ , cm	40	45	50	55	60	70	80
beam width $b$ , cm	15	20	20	25	25	30	30

Table 3

Cross-sectional sizes accepted

plate thickness cm	$h'_f = 9$ cm
the cross-sectional of the secondary beam	$h_2 \times b_2 = 50 \times 20$ cm
the cross-sectional of the head beam	$h_1 \times b_1 = 70 \times 30$ cm

## 2. PLATE CALCULATION AND DESIGN

Reinforced concrete plates are flat structures whose thickness  $h'_f$

is much smaller than width  $b$  and length  $l$ .

Initial data:

- a) heavy-weight concrete class C20; ( $f_{cd} = 11,5$  MPa);
- b) the plate is reinforced with welded fabric with transverse working armature (in the accepted variant, class A400C;  $f_{yd} = 365$  MPa, ( $\varnothing 6, 8$ mm));
- c) live load,  $v_n = 12$  kN / m<sup>2</sup>;
- d) reliability factor for the purpose of the building  $\gamma_n = 0,95$ ;
- e) weight of floor with preparation  $q_f = 1,2$  kN / m<sup>2</sup>;
- g) load safety factor  $\gamma_f = 1,2$ .

## 2.1. Development of structural scheme

The plates of the beam overlap lay on the secondary and head beams are plates supported on the contour and calculated in the direction of the shorter side. When calculating the plates, consider a strip 1m wide that rests on the walls (ending supports) and secondary beams (medium supports) (see Fig. 4).

The structural scheme of the plate is a continuing multi-span beam, which is loaded with a uniformly distributed load with intensity  $q$ . Load calculation is given in Section 5.3 of the methodical instructions.

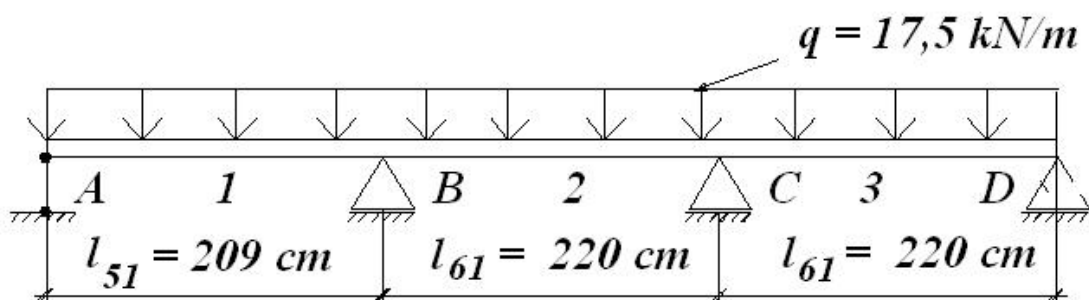


Fig. 5. The structural scheme of the plate

## 2.2. Determination of the estimated spans of the plate

The plate is pinched in the brick wall by the size of  $c = 12$  cm



(Fig. 6)

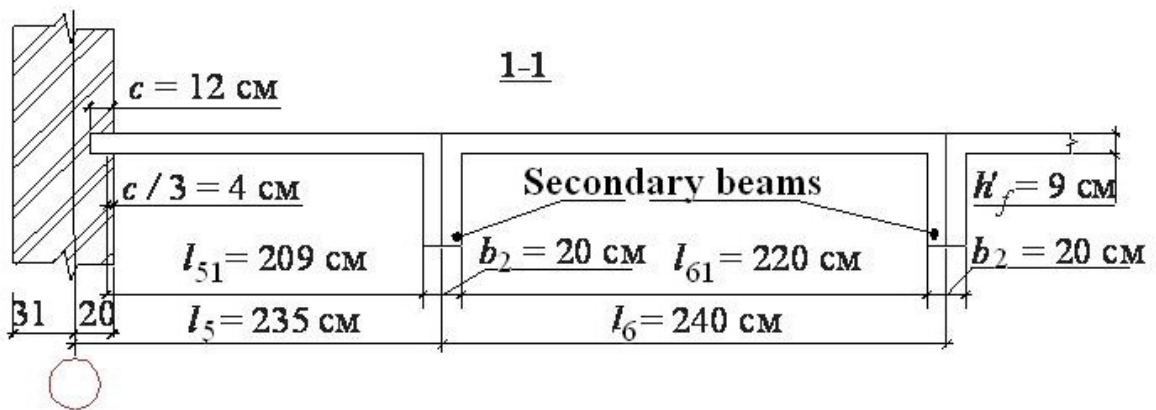


Fig. 6. To determine the estimated spans of the plate

Determination of the length of the calculated ending spans

$$l_{51} = l_5 - \frac{b_2}{2} - 20 + \frac{c}{3} = 235 - \frac{20}{2} - 20 + \frac{12}{3} = 209 \text{ cm},$$

where  $l_5 = 235$  cm – is the geometric length of ending span of the plate;

$b_2 = 20$  cm - width of the ribs of the secondary beam;

$c = 12$  cm – length of support of plate on the wall

Determination of estimated mean spans:

$$l_{61} = l_6 - b_2 = 240 - 20 = 220 \text{ cm},$$

where  $l_6 = 240$  cm is the geometric mean span of the plate;

$l_{61}$  is distance between the faces of secondary beams.

### 2.3. Determination of load acting on a plate

The calculations are reduced to table. 6. The design load per 1m of the plate is equal to the load per  $1\text{m}^2$ , because the width of the design strip of the plate is 1m. The density of heavy concrete is accepted  $\rho = 25 \text{ kN} / \text{m}^3$

**Table 6**

The load acting on the plate

No	Type of load	Characteristic load, kN	Load safety factor $\gamma_f$	Designed load, kN
	<b>Dead load</b>			
1	Reinforced concrete plate, $h'_f \cdot \rho = 0,09 \cdot 25 \cdot 1 \cdot 1 = 2,25$	2,25	1,1	2,47
2	Flooring $q_f = 1,2$	1,2	1,3	1,56
	Total dead load	$g_n = 3,45$		$g_1 = 4,03$
	<b>Temporary load</b>			
3	Live load, $v_n = 12$	$v_n = 12$	1,2	$v_1 = 14,4$

Total design load per  $1\text{m}^2$  of plate, taking into account the factor of reliability by purpose  $\gamma_n = 0,95$ :

$$q = g_1 \cdot \gamma_n + v_1 \cdot \gamma_n = 4,03 \cdot 0,95 + 14,4 \cdot 0,95 = 17,5 \text{ kN/m}$$

#### 2.4. Calculation of bending moments

The design efforts with regard to their redistribution due to plastic deformation of concrete are determined as follows:

- in the ending spans:

$$M_1 = \frac{q \cdot l_{51}^2}{11} = \frac{17,5 \cdot 2,09^2}{11} = 6,95 \text{ kN m};$$

- on supports B (first intermediate supports):

$$M_B = -\frac{q \cdot \left(\frac{l_{51} + l_{61}}{2}\right)^2}{11} = -\frac{17,5 \cdot \left(\frac{2,09 + 2,2}{2}\right)^2}{11} = -7,32 \text{ kN m};$$

- at mean spans:

$$M_2 = M_3 = \frac{q \cdot l_{61}^2}{16} = \frac{17,5 \cdot 2,20^2}{16} = 5,29 \text{ kN m};$$

-at intermediate supports:

$$M_C = -\frac{q \cdot l_{61}^2}{16} = \frac{17,5 \cdot 2,20^2}{16} = -5,29 \text{ kN m}.$$

The plot of the bending moments in the plate is shown in Fig.

7.

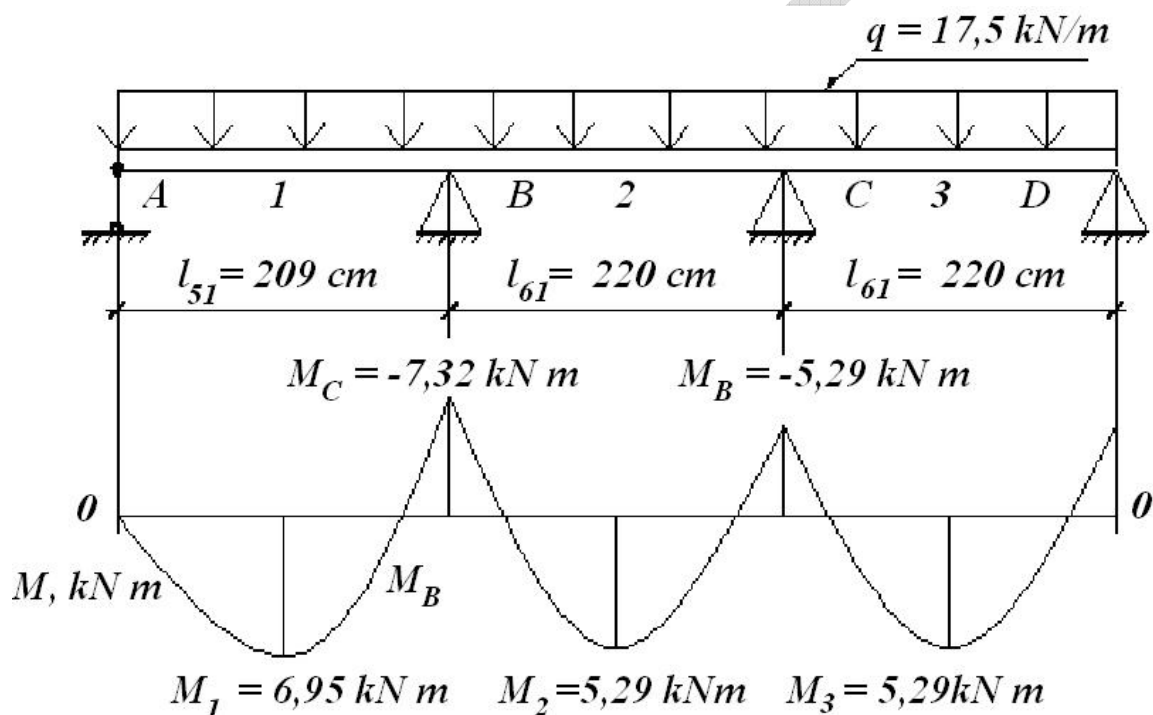


Fig. 7. Plot of bending moments in the plate

The shearing forces on the edges of the supports do not determine, because the condition of strength is always fulfilled.

## 2.5. Determination of plate thickness

The thickness of the plate previously taken to determine its weight is specified taking into account the action of the maximum bending moment  $M_B = 7,32 \text{ kN m}$ .

The effective height of the section is determined by the formula:

$$d = \sqrt{\frac{M}{\alpha_m \cdot b \cdot f_{cd}}},$$

where  $M = M_B = 732 \text{ kN cm}$  is the largest bending moment in the plate;

$b = 100 \text{ cm}$  - the estimated width of the plate.

To determine the tabular coefficient  $\alpha_m$ , it is necessary to find the effective height of the compressed zone of concrete  $\xi$ , which is calculated by the formula:

$$\xi = c \frac{f_{yd}}{f_{cd} \cdot 100\%}$$

where  $f_{cd} = 11,5 \text{ MPa}$  is the calculated compressive strength of concrete (for class C20);

$f_{yd} = 365 \text{ MPa}$  - calculated resistance of reinforcement to tensile (for class A400C);

The optimal reinforcement factor for plates supported on 4 sides is  $c = 0,3 \dots 0,6$ . We accept  $c = 0,5$ .

So that:

$$\xi = c \frac{f_{yd}}{f_{cd} \cdot 100\%} = 0,5 \frac{365}{11,5 \cdot 100\%} = 0,19.$$

Using the table of coefficients for calculations of bending elements reinforced with single reinforcement by size, we find the corresponding coefficient  $\alpha_m = 0,14$  according to table. 4, appendix.

Required working height of a section of a plate:

$$d = \sqrt{\frac{M}{\alpha_m \cdot b \cdot f_{cd}}} = \sqrt{\frac{732 \cdot (10)}{0,14 \cdot 100 \cdot 11,5}} = 6,74 \text{ cm.}$$

Full plate thickness:

$$h'_f = d + \frac{d_s}{2} + a = 6,74 + \frac{0,8}{2} + 1,0 = 8,14 \text{ cm}$$

where  $d_s = 8 \text{ mm} = 0,8 \text{ cm}$  - diameter of rods of working armature;

$a = 1,0$  cm - the minimum protective layer of concrete.  
 Assign the total thickness of the plate with a multiplicity of 1 cm.  
 Accept  $h'_f = 8,0$  cm.

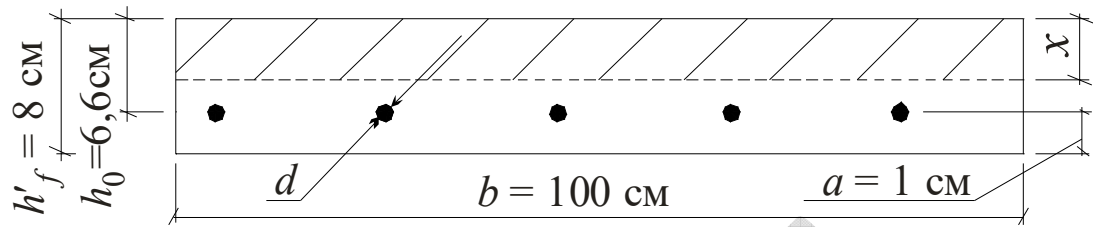


Fig. 8. Cross-section of the plate

Specify the effective height of the cross section:

$$d = h'_f - \frac{d}{2} - a = 8 - \frac{0,8}{2} - 1,0 = 6,6 \text{ cm.}$$

## 2.6. Determination of the cross area of the longitudinal reinforcement

For the perception of bending moments in flat areas flat welded nets fabrics with transverse working armature of class A400C are installed, with resistance  $f_{yd} = 365$  MPa.

The selection of fittings is summarized in Table. 7, 8 and its arrangement is shown in the diagrams (Figs. 9, 10).

## 2.7. Instructions for plate construction

In the floor plates of sections B, which are completely framed along the contour of the main and minor beams, we reduce the amount of bending moments by 20%.

It is recommended that the ribbed plate be reinforced with welded nets. If the diameter of the working fittings 3, 4, 5mm, then take roll nets with longitudinal working fittings, which unfold in the direction of the main beams.

In the spans, the grids are laid on the bottom of the plate, and on the supports, over the minor beams are transferred to the upper

zones of the plate. The bending of the nets in the upper zone is at the distance of the span from the axis of the support. The main grid C1 (section A) is selected by the magnitude of the torque acting in the middle spans. In extreme spans, an additional C2 grid is laid.

The main grid C3 (section B) is selected by the value of the torque, which is reduced by 20%, and which operates in the middle spans. In extreme spans, an additional C4 grid is laid (Fig. 9).

Table 7

### Calculation of reinforcements section A (see fig. 3)

Element	$M$ , κN·cm	$\alpha_m = \frac{M}{b d^2 f_{cd}}$	$\zeta$	Required reinforcement $A_s = \frac{M}{f_{yd} d \zeta}$ , cm <sup>2</sup>	Adopted reinforcement	
					Number and types of welded fabric	$A_s$ , cm <sup>2</sup>
Span1	695	$\frac{6,95 \cdot 100 \cdot 10}{100 \cdot 6,6^2 \cdot 11,5} = 0,139$	0,924	$\frac{6,95 \cdot 100 \cdot 10}{365 \cdot 6,6 \cdot 0,924} = 3,12$	$\frac{4Bp-I-250}{8A400C-150}$	3,36
Support B	732	$\frac{7,32 \cdot 100 \cdot 10}{100 \cdot 6,6^2 \cdot 11,5} = 0,146$	0,92	$\frac{7,32 \cdot 100 \cdot 10}{365 \cdot 6,6 \cdot 0,92} = 3,31$	$\frac{4Bp-I-250}{8A400C-150}$	3,36
Span2	530	$\frac{5,30 \cdot 100 \cdot 10}{100 \cdot 6,6^2 \cdot 11,5} = 0,106$	0,944	$\frac{5,30 \cdot 100 \cdot 10}{365 \cdot 6,6 \cdot 0,944} = 2,33$	$\frac{4Bp-I-250}{8A400C-200}$	2,51
Support C	530	$\frac{5,30 \cdot 100 \cdot 10}{100 \cdot 6,6^2 \cdot 11,5} = 0,106$	0,944	$\frac{5,30 \cdot 100 \cdot 10}{365 \cdot 6,6 \cdot 0,944} = 2,33$	$\frac{4Bp-I-250}{8A400C-200}$	2,51

### 3. SECONDARY BEAM CALCULATION AND DESIGN

Initial date:

- a) heavy-weight concrete class C20; ( $f_{cd} = 11,5 \text{ MPa}$ );
- b) in span the secondary beam is reinforced with welded carcasses with longitude working armature of class A400C;  $f_{yd} = 365 \text{ MPa}$ ,  $f_{ywd} = 290 \text{ MPa}$ ;
- c) At support the secondary beam is reinforced with welded fabric with transverse working armature (in the accepted variant, class A240C;  $f_{yd} = 365 \text{ MPa}$ , ( $\emptyset 6$ , 8mm));
- d) live load,  $v_n = 12 \text{ kN / m}^2$ ;
- e) reliability factor for the purpose of the building  $\gamma_n = 0,95$ ;
- f) weight of floor with preparation  $q_f = 1,2 \text{ kN / m}^2$ .

#### 3.1. Development of structural scheme

The design structural scheme of the secondary beam is a continuous six-span beam. It is supported by main beams and its ends lay on the brick walls (Fig. 3). The beam is loaded with a uniformly distributed load  $q$ . The load calculation is given in Section 6.3.

If the number of spans exceeds 5, then the beam is calculated as five-span (see Fig. 11).

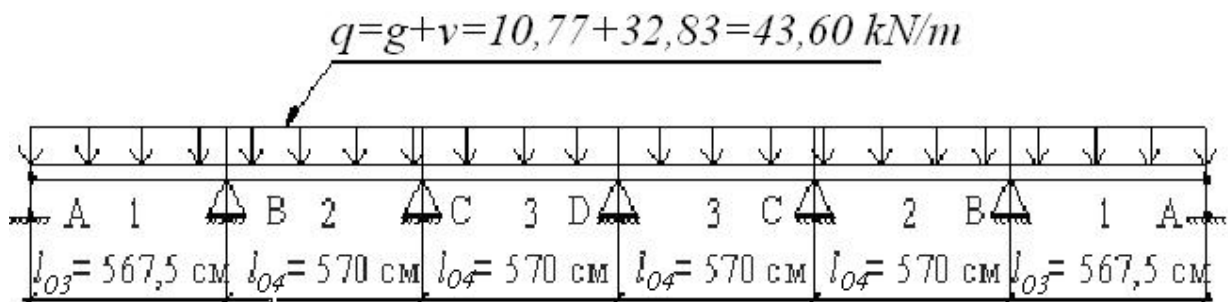


Fig. 11. Structural scheme of secondary beam

#### 6.2. Determination of the designed spans of the secondary beam

The secondary beams are walled up by a  $c = 25 \text{ cm}$ .

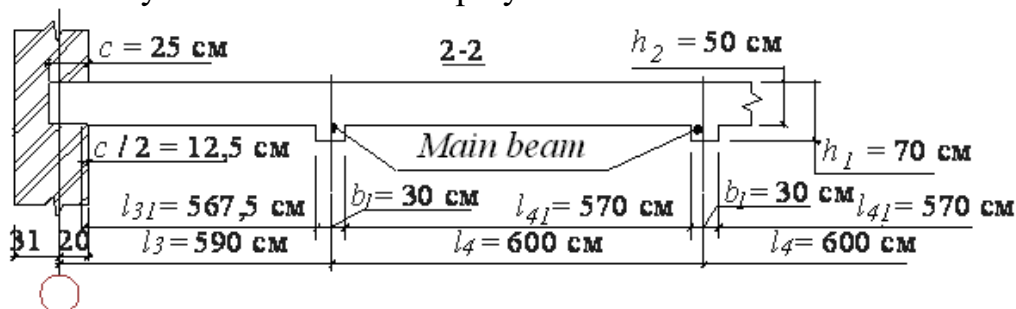


Fig. 12. Determination of the designed spans of secondary beam



Designed spans are evaluated taking into account previously accepted cross-sectional dimensions of the secondary and main beams. (Table 2).

Determination of the designed spans :

$$l_{31} = l_3 - 20 - \frac{b}{2} + \frac{c}{2} = 590 - 20 - \frac{30}{2} + \frac{25}{2} = 567,5 \text{ см,}$$

де  $l_3 = 590$  см – last geometric span of the secondary beam;

$b_1 = 30$  см – width of the ribs of the main beam;

$c = 25$  см – the distance of the support of the beam on the wall.

Determination of designed transitional spans:

$$l_{41} = l_4 - b_1 = 600 - 30 = 570 \text{ см,}$$

де  $l_4 = 600$  – transitional geometric span of the secondary beam;

$l_{41}$  – the distance between the faces of the main beams.

### 3.3. Determination of the load on the beam

Evenly distributed load consists of a constant (weight of the floor, plate, ribs of the secondary beam) and the temporary load  $v_n = 12$  kN/m. The load is collected from the loading area, the width of which is limited by the larger span of the plate  $l_6 = 2,40$  m (see Fig. 4).

Table 9

Визначення навантаження на другорядну балку

№	Type of load	Characteristic load, кN/м	Load safety factor $\gamma_f$	Designed load, кN/м
Dead load				
1	Flooring $q_{II} \cdot l_6 = 1,2 \cdot 2,40 = 2,88$	2,88	1,3	3,74
2	Reinforced concrete plate, $h'_f \cdot l_6 \cdot 1 \cdot \rho =$ $0,08 \cdot 2,40 \cdot 1 \cdot 25 = 4,8$	4,8	1,1	5,28
3	Ribs of the secondary beam $(h_2 - h'_f) \cdot b_2 \cdot 1 \cdot \rho =$ $= (0,50 - 0,08) \cdot 0,2 \times$ $\times 25 \cdot 1 = 2,1$	2,1	1,1	2,31
	Total dead load	$g_i = 9,78$		$g_1 = 11,33$
Temporary load				
4	$v_n \cdot l_6 = 12 \cdot 2,40 = 28,8$	$v_n = 28,8$	1,2	$v_1 = 34,56$

Total design load per 1m of secondary beam, taking into account the factor of reliability by purpose  $\gamma_n = 0,95$ :

$$g = g_1 \cdot \gamma_n = 11,33 \cdot 0,95 = 10,77 \text{ kN/M,}$$

$$v = v_1 \cdot \gamma_n = 34,56 \cdot 0,95 = 32,83 \text{ kN/M,}$$

$$q = g + v = 10,77 + 32,83 = 43,60 \text{ kN/M.}$$

#### 6.4. Calculation of bending moments in secondary beam

The design of the secondary beam is related to the construction of the bending moment curve, whose ordinates are determined by the formula:

$$M = \pm\beta \cdot (g + v) \cdot l_0^2,$$

where  $\beta$  – e coefficient taken depending on the cross section of the beam and the load ratio  $\frac{v}{g}$ , from Annex Table. 6;

$$(g + v) = (10,77 + 32,83) = 43,60 \text{ kN/M} \quad \text{- total design load;}$$

$l_0$  - the estimated span length, which defines the bending moments. The definition of bending moments for relation  $\frac{v}{g} = \frac{32,83}{10,77} \approx 3$ , is given in Table. 10.

Construction of the diagrams  $M$  and  $Q$  shown in Fig. 13.

Table 10

#### Calculation of bending moments in secondary beam

span	section	Coefficients		$(g + v) \cdot l_0^2$	Bending moments, kNm	
		$\beta$	$-\beta$		$M_{\max}$	$M_{\min}$
1 span	1	0,065	-	$43,6 \times 5,675^2 = 1404,14$	91,25	-
	2	0,09	-		126,35	-
	2'	0,091	-		127,75	-
	3	0,075	-		105,29	-
	4	0,02	-		28,08	-
Support. B	5	-	-0,0715	$43,6 \times 5,687^2 = 1410,34^*$	-	-100,82
2 span	6	0,018	-0,035	$43,6 \times 5,7^2 = 1416,54$	25,50	-49,57
	7	0,058	-0,016		82,14	-22,66
	7'	0,0625	-		88,51	-
	8	0,058	-0,014		82,14	-19,83
	9	0,018	-0,029		25,50	-41,07
Support. C	10	-	-0,0625	$43,6 \times 5,7^2 = 1416,54$	-	-88,51
3 span	11	0,018	-0,028	$43,6 \times 5,7^2 = 1416,54$	25,50	-39,65
	12	0,058	-0,01		82,14	-14,16
	12'	0,0625	-		88,52	-

- on support B the design span is defined as the arithmetic mean value  $\frac{l_{31} + l_{41}}{2}$

### 3.5. Calculation of shear forces

$$Q_A = 0,4 \cdot (g + v) \cdot l_{31} = 0,4 \cdot 43,6 \cdot 5,675 = 98,95 \text{ кН};$$

$$Q_B^r = -0,6 \cdot (g + v) \cdot l_{31} = -0,6 \cdot 43,6 \cdot 5,675 = -148,42 \text{ кН};$$

$$Q_B^l = -Q_C^r = Q_C^l = \pm 0,5 \cdot (g + v) \cdot l_{41} = 0,5 \cdot 43,6 \cdot 5,7 = \pm 124,23 \text{ кН}.$$

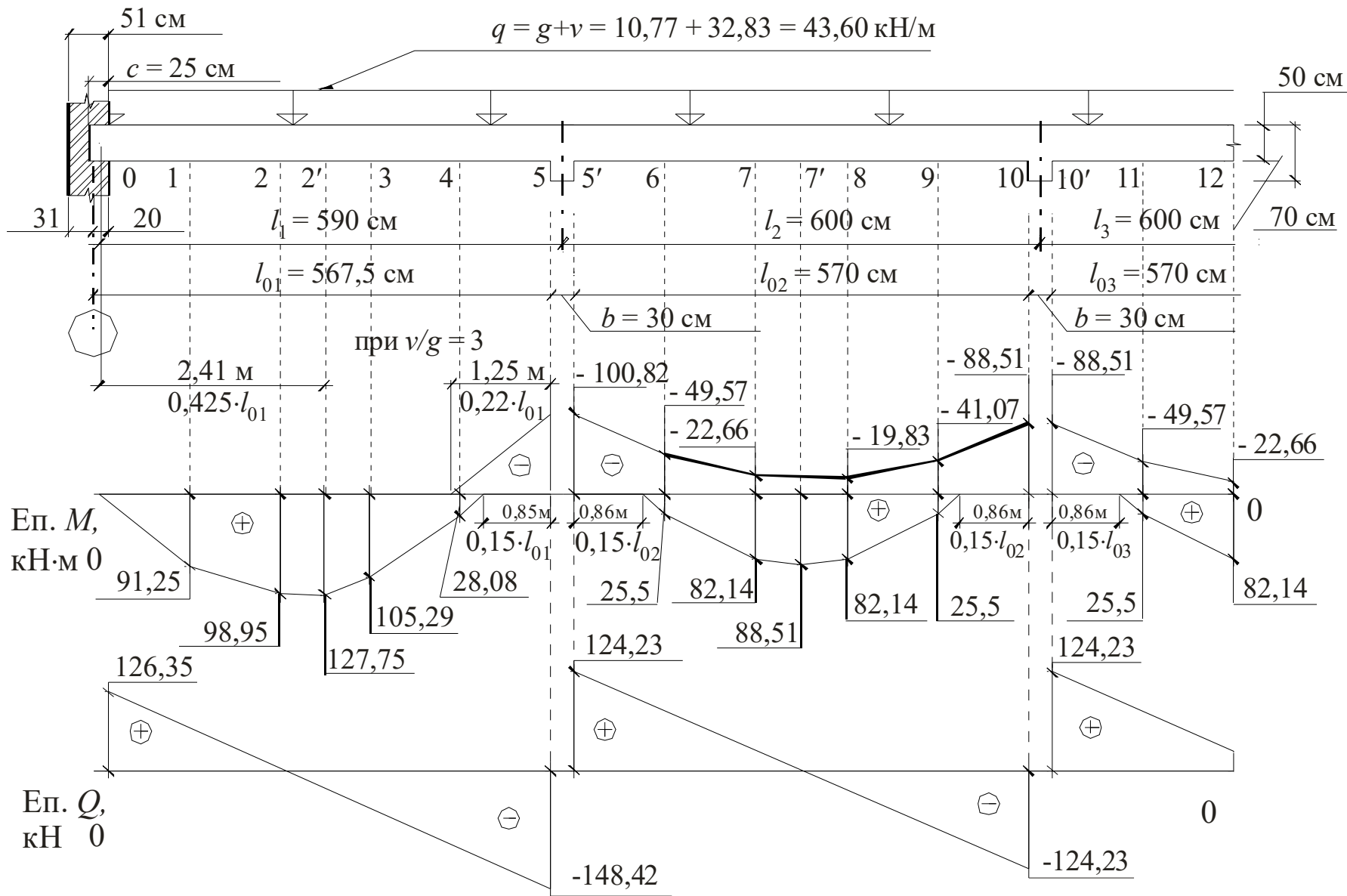


Fig. 13. Diagrams of bending moments and shear forces for secondary beam ( $M - \text{кНм}$ ,  $Q - \text{кН}$ ).

### 3.6. Sizing the cross section of a secondary beam

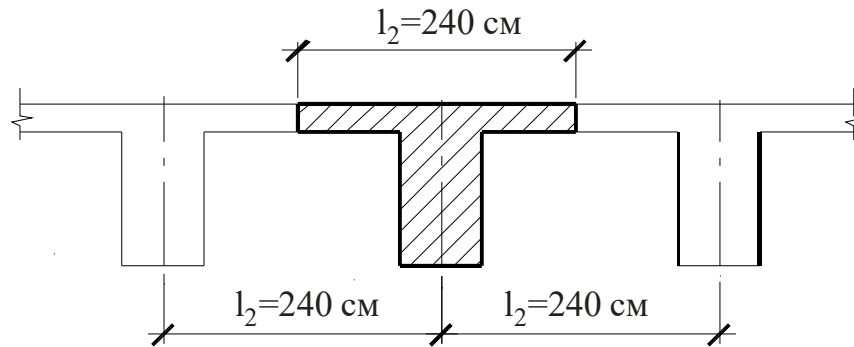


Fig.14. Cross-section of floor in transversal direction

Effective height of cross-section of secondary beam:

$$d_2 = \sqrt{\frac{M}{\alpha_m \cdot f_{cd} \cdot b_w}},$$

where  $M = 127,78 \text{ kN}\cdot\text{cm}$  – greatest bending moment at the beam;

$\alpha_m = 0,241$  – determined by the optimal value  $\xi = 0,3 \dots 0,4$ ;

$$\alpha_0 \rightarrow \xi = \frac{x}{h_0} = 0,35;$$

$f_{cd} = 11,5 \text{ MPa}$  – designed strength for compression for heavy weight concrete of class B20;

$b_2 = 20 \text{ cm}$  – the width of ribs of secondary beam.

So,

$$d_2 = \sqrt{\frac{M}{\alpha_m \cdot f_{cd} \cdot b_w}} = \sqrt{\frac{127,78 \cdot 100 \cdot 10}{0,241 \cdot 11,5 \cdot 20}} = 48,01 \text{ cm}.$$

The whole height of secondary beam  $h = h_0 + a = 48,01 + 5 = 53,01 \text{ cm}$

here  $a = 5 \text{ cm}$  is thickness of concrete protective layer.

We refined the sizes of cross section for secondary beam

We accept height of secondary beam  $h_2 = 50 \text{ cm}$ , the rib width  $b_2 = 20 \text{ cm}$ .

Specify effective height for secondary beam:

a) the rods are placed in two rows:

$$d_{21} = h_2 - a = 50 - 5 = 45 \text{ cm};$$

b) welded fabrics are placed on a support:

$$d_{22} = h_2 - d'_1 = 50 - 3 = 47 \text{ cm};$$

b) welded fabrics are placed on a support:

$$d_{23} = h_2 - d'_2 = 50 - 5 = 45 \text{ cm}.$$

Recommended beam section sizes:

beam height $h$ , cm	40	45	50	55	60	70	80
beam width $b$ , cm	15	20	20	25	25	30	30

The flange width is determined by the formula:

$$b_{eff} = b_2 + b_{eff1} + b_{eff2}$$

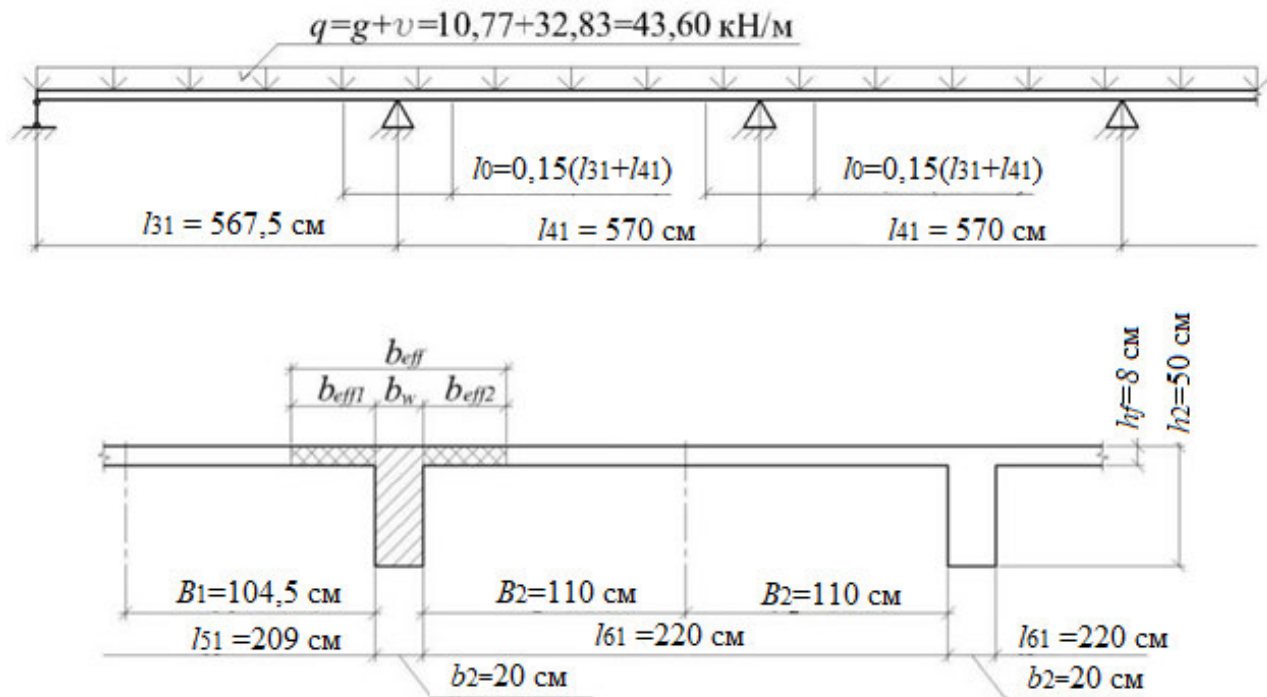


Fig. 15. Determination the conditional span of the secondary beam  $l_0$  and the width of the shelf  $b_{eff}$

The width of the shelf is determined depending on the conditional span

$$l_0 = 0,15(l_{31} + l_{41}) = 0,15(567,5 + 570) = 170,6 \text{ cm}$$

and, accordingly, the width of the overhangs of the shelf

$$b_{eff1} = 0,2B_1 + 0,1l_0 \leq 0,2l_0,$$

$$b_{eff2} = 0,2B_2 + 0,1l_0 \leq 0,2l_0$$

where  $B_1 = 0,5 \cdot l_{51} = 0,5 \cdot 209 = 104,5 \text{ cm}$  and  $B_2 = 0,5l_{61} = 0,5 \cdot 220 = 110 \text{ cm}$

If the condition is not met, we accept  $b_{eff1}$  and  $b_{eff2} = 0,2l_0$

As follows,

$$b_{eff1} = 0,2B_1 + 0,1l_0 = 0,2 \cdot 104,5 + 0,1 \cdot 170,6 = 379,6 \text{ cm} >$$

$$> 0,2l_0 = 0,2 \cdot 170,6 = 34,12 \text{ cm}$$

The condition is not met, we accept  $b_{eff1} = 34,12 \text{ cm}$ .

$$b_{eff2} = 0,2B_2 + 0,1l_0 = 0,2 \cdot 110 + 0,1 \cdot 170,6 = 390,6 \text{ cm} >$$

$$> 0,2l_0 = 0,2 \cdot 170,6 = 34,12 \text{ cm}$$

The condition is not met, we accept  $b_{eff2} = 34,12 \text{ cm}$ .

Design width of flange

$$b_{eff} = b_2 + b_{eff1} + b_{eff2} = 20 + 34,1 + 34,1 = 88,24 \text{ cm}.$$

We accept  $b_{eff} = 88 \text{ cm}$ .

### 3.7. Determination of the calculated designed shape of secondary beam cross-section

Determination of neutral line position:

$$M_f = b_{eff} h_f f_{cd} \cdot (d_{21} - 0,5 \cdot h_f) = 88 \cdot 8 \cdot \frac{11,5}{10} \cdot (45 - 0,5 \cdot 8) = 33193 \text{ kNm} =$$

$$= 331,93 \text{ kNm}$$

Since  $M = 127,75 \text{ kNm} \leq M_f = 331,93 \text{ kNm}$  the cross section is calculated as rectangular with width  $b'_f = 88 \text{ cm}$  and height  $h_2 = 50 \text{ cm}$ .

### 3.8. Determination of the cross-section area of the longitudinal reinforcement

The reinforcement of the secondary beam in the spans of two flat welded frames is envisaged. Each frame has two lower rods and one upper of the class A400C fittings with the designed strength  $R_s = 365 \text{ MPa}$ .

Attention

b) in span the secondary beam is reinforced with welded carcasses with longitude working armature of class A400C;  $R_s = 365 \text{ MPa}$ ,  $R_{sw} = 290 \text{ MPa}$ ; (see your form)

On the supports B and C for the perception of negative bending moments of the beam reinforced by two flat welded fabric with transverse working armature class A400C.

The selection of the number and diameter of the reinforcing rods is performed according to Table. 3 of Applications.

Selection of type and area of cross-section the working armature of flat welded fabric is performed according to Table. 4 of Application.

Table 11

**Calculation of longitude reinforcement of secondary beam**

Element of beam	Point	$M_{Ed}$ кН М	$b$	Effective height, cm $d$	$\alpha_m = \frac{M}{bd^2 f_{cd}}$	$\zeta$	Required reinforcement $A_s = \frac{M}{f_{yd} d \zeta}$	Adopted reinforcement	
								Number of reinforcement	Actual cross-area $A_s, \text{cm}^2$
Span 1	2' max	127,78	$b_{eff} = 88$	$d_{21} = 45$	$\frac{127,78 \cdot 100 \cdot 10}{88 \cdot 45^2 \cdot 11,5} = 0,0624$	0,969	$\frac{127,78 \cdot 100 \cdot 10}{365 \cdot 45 \cdot 0,969} = 8,03$	4Ø16A400C	8,04
Span 2	7' max	88,54	$b_{eff} = 88$	$d_{21} = 45$	$\frac{88,54 \cdot 100 \cdot 10}{88 \cdot 45^2 \cdot 11,5} = 0,0432$	0,978	$\frac{88,54 \cdot 100 \cdot 10}{365 \cdot 45 \cdot 0,978} = 5,51$	4 Ø14A400C	6,16
	7 min	22,66	$b_2 = 20$	$d_{22} = 47$	$\frac{22,16 \cdot 100 \cdot 10}{20 \cdot 45^2 \cdot 11,5} = 0,044$	0,978	$\frac{22,16 \cdot 100 \cdot 10}{365 \cdot 45 \cdot 0,978} = 1,35$	2 Ø12A400C	2,26
Support B	5 min	100,84	$b_2 = 20$	$d_{23} = 45$	$\frac{100,82 \cdot 100 \cdot 10}{20 \cdot 45^2 \cdot 11,5} = 0,216$	0,876	$\frac{100,82 \cdot 100 \cdot 10}{365 \cdot 45 \cdot 0,876} = 7,00$ Required cross-area of working road in 1m of welded fabric $A_{s1} = \frac{A_s}{2 \cdot l_6} = \frac{7,00}{2 \cdot 2,4} = 1,46$	2 welded fabric 4Bp-I-250 6A400C-150 )	9,07 (1,89)



Table 11. Continuation

Support C	10 min	88,52	$b_2 = 20$	$d_{23} = 45$	$\frac{88,52 \cdot 100 \cdot 10}{20 \cdot 45^2 \cdot 11,5} = 0,19$	0,894	$\frac{88,52 \cdot 100 \cdot 10}{365 \cdot 45 \cdot 0,894} = 6,03$ Required cross-area of working rod in 1m of welded fabric $A_{s1} = \frac{A_s}{2 \cdot l_6} = \frac{6,03}{2 \cdot 2,4} = 1,26$	<i>2 welded fabric</i> $\frac{4Bp - I - 250}{6A400C - 200}$	6,77 (1,41)
-----------	--------	-------	------------	---------------	--	-------	---	--	----------------

**Table 12**

Calculation of bearing capacity of the secondary beam  $M_u$

Element of beam	$b$	Effective height $d$	Working reinforcement		$\xi = \frac{A_s}{0,8 \cdot b \cdot d} \cdot \frac{f_{yd}}{f_{cd}}$	$\zeta$	Bearing capacity $M_u = A_s d \zeta f_{yd}$ , кНсМ	$M_{Ed}$ кНсМ
			Number of reinforcement	$A_s$ , мм <sup>2</sup>				
Span 1	$88 b_{eff}$	$d_{21} = 45$	4Ø16A400C	8,04	$\frac{8,04}{0,8 \cdot 88 \cdot 45} \cdot \frac{365}{11,5} = 0,0805$	0,968	$8,04 \cdot 45 \cdot 0,968 \cdot \frac{365}{10} = 12783$	12778
Span 2	$88 b_{eff}$	$d_{21} = 45$	4Ø 14A400C	6,16	$\frac{6,16}{0,8 \cdot 88 \cdot 45} \cdot \frac{365}{11,5} = 0,0617$	0,975	$6,16 \cdot 45 \cdot 0,975 \cdot \frac{365}{10} = 9865$	8854
	$b_2 = 20$	$d_{22} = 47$	2Ø12A400C	2,26	$\frac{2,26}{0,8 \cdot 20 \cdot 47} \cdot \frac{365}{11,5} = 0,095$	0,962	$2,26 \cdot 47 \cdot 0,962 \cdot \frac{365}{10} = 3729$	2266
Support B	$b_2 = 20$	$d_{23} = 45$	2 welded fabric $\frac{4Bp - I - 250}{6A400C - 150}$	9,07	$\frac{9,07}{0,8 \cdot 20 \cdot 45} \cdot \frac{365}{11,5} = 0,3998$	0,840	$9,07 \cdot 45 \cdot 0,840 \cdot \frac{365}{10} = 12513$	10084
Support C	$b_2 = 20$	$d_{23} = 45$	2 welded fabric $\frac{4Bp - I - 250}{6A400C - 200}$	6,77	$\frac{6,77}{0,8 \cdot 20 \cdot 45} \cdot \frac{365}{11,5} = 0,298$	0,880	$6,77 \cdot 45 \cdot 0,880 \cdot \frac{365}{10} = 9785$	8852

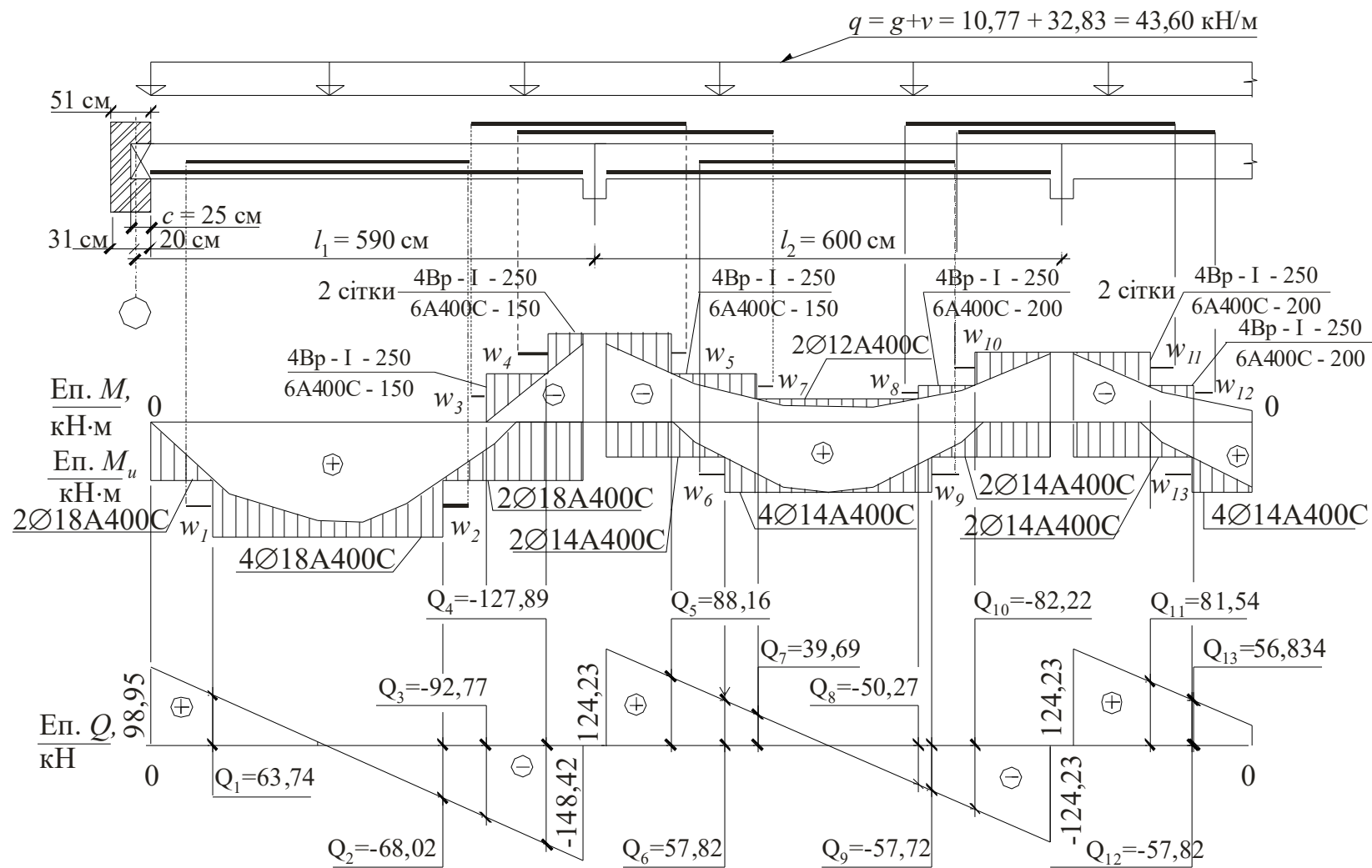


Fig. 17. Plots of  $M_{Ed} - M_u$  and  $V_{Ed} - V_u$  of secondary beam

### 3.8 Calculation of the strength of the secondary beam on inclined sections

In each span, the beam is reinforced with two welded frames with transverse reinforcement class A240C with the calculated tensile strength  $f_{ywd} = 170$  MPa.

At the ending sections of the beam, each lof  $0,25l$  length, there are large enough transverse forces and the step of the transverse reinforcement is assigned taking into account the following conditions:

- at the height of the beam:

$$h < 450 \text{ MM} \Rightarrow s_{1w} \leq \frac{1}{2}h, \text{ in addition } s_{1w} < 150 \text{ MM};$$

$$h > 450 \text{ MM} \Rightarrow s_{1w} \leq \frac{1}{3}h, \text{ in addition } s_{1w} < 500 \text{ MM}.$$

In the example under consideration,  $h_2 = 500$  mm, respectively, the step of the clamps take  $s_{1w} = 150$  mm. Recommended steps of transverse vertical reinforcement 100; 125; 150; 250 mm.

The maximum step of the transverse vertical reinforcement in accordance with current regulations may not exceed  $s_{w,\max} = 0,75d$  ( $d$  - is the working height of the section).

In the middle part of the span of the beam, where the transverse forces are insignificant, the step of the transverse rods  $s_{2w}$  is assigned taking into account the conditions:

$$s_{2w} \leq \frac{3}{4}h, \text{ in addition } s_{2w} \leq 500 \text{ MM}.$$

we accept  $s_{2w} = 250$  mm.

In all cases, it is recommended to take a step of transverse vertical rods multiples of 50 mm, with rounding in the smaller direction.

The need to install the calculated vertical transverse reinforcement is determined by checking y

$$V_{Ed} \leq V_{Rd,c} \text{ or } v_{Ed} \leq v_{Rd,c}$$

where  $V_{Ed} = 148,46$  κH – value of designed shear force (support B left )

$V_{Rd,c} = H_{Ed} b_w d$  - transverse force, which can be perceived only by the concrete cross-section of the rib of the secondary beam.

From here

$$H_{Ed} = \frac{V_{Rd,c}}{b_w d_{21}} = \frac{148,46 \cdot 10}{20 \cdot 45} = 1,65 \text{ MPa} - \text{ design shear stress in cross section.}$$

Determine the design strength (stress) on the shear of the section reinforced with

longitudinal reinforcement:

$$H_{Rd,c} = \frac{C_{Rd,c}}{z_{ct}} \cdot k \sqrt[3]{100 \cdot c_1 \cdot f_{ck}},$$

where  $C_{Rd,c} = 0,18 \text{ MPa}$  is the minimum value (normalized) of shear strength of concrete (Appendix 1, Table 3);

$z_{ct} = 1,5$  - coefficient of reliability for concrete in tensile work ;

$k = 1 + \sqrt{\frac{20}{d_{21}}} \leq 2,0$  - coefficient that takes into account the influence of section

height.

So

$$k = 1 + \sqrt{\frac{20}{d_{21}}} = 1 + \sqrt{\frac{20}{45}} = 1,44 \leq 2,0$$

When  $k > 2,0$  we take  $k = 2$ ;

So

$$c_1 = \frac{A_{sB}}{b_2 d_{22}} = \frac{8,32}{20 \cdot 47} = 0,0088 \leq c_{\max} \leq 0,02$$

Determine the shear strength of concrete with shear, which is recommended to be determined by the expression:

$$H_{Rd,c} = \frac{C_{Rd,c}}{z_{ct}} \cdot k \sqrt[3]{100 \cdot c_1 \cdot f_{ck}} = \frac{0,18}{1,5} \cdot 1,44 \sqrt[3]{100 \cdot 0,0088 \cdot 15} = 0,4 \text{ MPa}$$

Due to the fact that  $H_{Ed} = 1.65 \text{ MPa} > H_{Rd,c} = 0.4 \text{ MPa}$ , it becomes necessary to reinforce the inclined sections with the calculated vertical transverse reinforcement.

Determine the factor of shear strength of concrete with crack, which is recommended to be determined by the expression:

$$h = 0,6 \cdot \left[ 1 - \frac{f_{ck}}{250} \right] = 0,6 \cdot \left[ 1 - \frac{15}{250} \right] = 0,546 < 0,6$$

The area of the vertical transverse reinforcement is determined under the condition that the angle of inclination of the compressed strips (strips between possible inclined cracks) of fictitious struts of the truss model can take any values within  $21,8^\circ \leq u \leq 45^\circ$ . This angle depends on the maximum possible shear strength of the concrete  $H_{Rd,\max}$ , which in turn depends on the design compressive strength of concrete  $f_{cd}$ .

Thus the maximum shear strength of concrete (shear) at  $\cot u = 2,5$ ;  $\text{tg } u = 0,4$ ;  $\cot^2 u = 6,25$  (for  $u = 21,8^\circ$ ).

$$H_{Rd, \max} = f_{cd} \cdot h \cdot \left( \frac{\cot u + \operatorname{tg} u}{1 + \cot^2 u} \right) = 11,5 \cdot 0,564 \left( \frac{2,5 + 0,4}{1 + 6,25} \right) = 2,59 \text{ MPa}$$

Due to the fact that at,  $\cot u = 2,5$ ,  $H_{Rd, \max} = 2,59 \text{ MPa} > H_{Ed} = 1,65 \text{ MPa}$ , the calculated area of the transverse vertical reinforcement at its step  $s_{1w} = 150 \text{ mm}$  will be:

$$A_{sw} = \frac{H_{Ed} \cdot s_{w1} \cdot b_w}{0,8 \cdot f_{ywd} \cdot \cot u} = \frac{1,65 \cdot 15 \cdot 20}{0,8 \cdot 170 \cdot 2,5} = 1,456 \text{ cm}^2.$$

Determine the calculated reinforcement coefficient of the transverse reinforcement

$$c_w = \frac{A_{sw}}{b_2 \cdot s_w} = \frac{1,46}{20 \cdot 15} = 0,0049 > c_{w, \min} = 0,0016$$

Take 2Ø10A240C with a step  $s_w = 150 \text{ mm}$ ,  $A_{sw} = 1.57 \text{ cm}^2$ .

If it is necessary to reduce the diameter of the transverse reinforcement, reduce the pitch of the specified reinforcement.

Example,

$$A_{sw} = \frac{H_{Ed} \cdot s_{w1} \cdot b_w}{0,8 \cdot f_{ywd} \cdot \cot u} = \frac{1,65 \cdot 10 \cdot 20}{0,8 \cdot 170 \cdot 2,5} = 0,982 \text{ cm}^2.$$

$c_w = \frac{A_{sw}}{b_2 \cdot s_w} = \frac{0,982}{20 \cdot 15} = 0,0033 > c_{w, \min} = 0,0016$ , we accept 2 Ø 8A240C with a step  $s_{w1} = 100 \text{ mm}$ ,  $A_{sw} = 1,01 \text{ cm}^2$ .

The area of the vertical transverse reinforcement in the middle part of the first span is determined by the value of the calculated transverse force at a distance of  $\approx 0,25l_{31} = 0,25 \cdot 5675 = 1420 \text{ mm}$  from the face B of the support. In our example,  $V_{Ed 0,25} = 85.67 \text{ kN}$ .

Accordingly, the calculated shear stress in cross section at a distance of 1420 mm is:

$$H_{Ed 0,25} = \frac{V_{Rd 0,25}}{b_w d_{21}} = \frac{85,67 \cdot 10}{20 \cdot 45} = 0,95 \text{ MPa}.$$

Determine the area of the vertical transverse reinforcement with a pre-accepted step  $s_{w2} = 250 \text{ mm}$ :

$$A_{sw} = \frac{H_{Ed 0,25} \cdot s_{w2} \cdot b_w}{0,8 \cdot f_{ywd} \cdot \cot u} = \frac{0,95 \cdot 25 \cdot 20}{0,8 \cdot 170 \cdot 2,5} = 1,39 \text{ cm}^2.$$

We accept 2 Ø 10A240C with a step  $s_{w2} = 250$  mm,  $A_{sw} = 1,57$  mm<sup>2</sup>.

Determine the magnitude of the transverse force, which is perceived by the adopted transverse vertical reinforcement in the frames of the extreme spans:

a) the supporting section of the beam:

first calculate the limit value of shear stresses

$$h_{u1} = \frac{0,8 \cdot A_{sw} \cdot f_{ywd} \cdot \cot u}{s_{w1} \cdot bw} = \frac{0,8 \cdot 1,57 \cdot 170 \cdot 2,5}{15 \cdot 20} = 1,78 \text{ MPa.}$$

The magnitude of the transverse force in the supporting areas

$$V_{u1} = h_u \cdot b_w \cdot d_{21} = 0,1 \cdot 1,78 \cdot 20 \cdot 45 = 160,4 \text{ kN.}$$

b) the middle part of the beam:

shear stress limit

$$h_{u2} = \frac{0,8 \cdot A_{sw} \cdot f_{ywd} \cdot \cot u}{s_{w2} \cdot bw} = \frac{0,8 \cdot 1,57 \cdot 170 \cdot 2,5}{25 \cdot 20} = 1,067 \text{ MPa.}$$

The magnitude of the transverse force in the middle of the beam

$$V_{u2} = h_u \cdot b_w \cdot d_{21} = 0,1 \cdot 1,067 \cdot 20 \cdot 45 = 96,03 \text{ kN.}$$

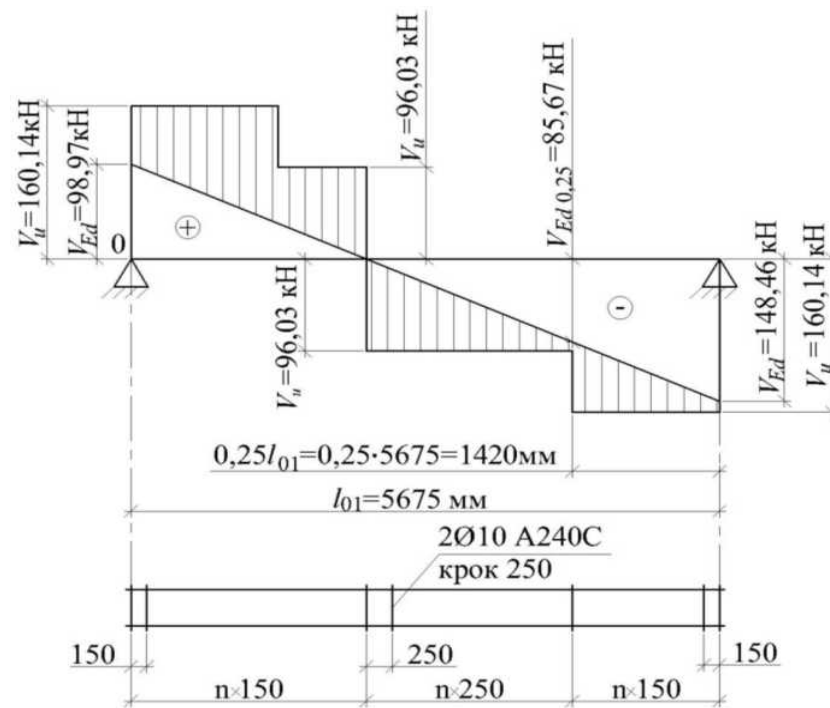


Fig. 16. Plot  $V_{Ed}$  and  $V_u$ , kN

The calculated values of the transverse forces and the arrangement of the frames of the middle spans in the given example practically coincide with the values of the calculated transverse forces and the arrangement of the extreme spans. Therefore, the

magnitude of the transverse force, which is perceived by the adopted transverse vertical reinforcement of the middle spans, is not determined



**Annex I**

Reference data for calculations

**Table 1**

**Characteristics of strength and deformability of concrete**

Characteristic	Concrete strength class										
	C10	C15	C20	C25	C30	C35	C40	C45	C50	C55	C60
$f_{ck,cube}$ , MPa	10	15	20	25	30	35	40	45	50	55	60
$f_{cm,cube}$ , MPa	13	19	25	32	38	45	51	58	64	71	77
$f_{ck,prism}$ , MPa	7,5	11	15	18,5	22	22,5	29	32	36	39,5	43
$f_{cd}$ , MPa	6,0	8,5	11,5	14,5	17	19,5	22	25	27,5	30	33
$f_{ctm}$ , MPa	1,2	1,6	1,9	2,2	2,6	2,8	3,0	3,2	3,5	3,8	4,1
$f_{ctk,0,05}$ , MPa	0,8	1,1	1,3	1,5	1,8	2,0	2,1	2,2	2,5	2,7	3,0
$E_{cm} \cdot 10^3$ , MPa	18	23	27	30	32,5	34,5	36	37,5	39	39,5	40
$E_{cd} \cdot 10^3$ , MPa	12,6	16,3	20	23	25	27	28,5	30,5	32	33	34
$\epsilon_{cl,ck}$ , (‰)	1,57	1,61	1,66	1,71	1,76	1,81	1,86	1,90	1,94	1,98	2,02
$\epsilon_{cl,cd}$ , ‰	1,56	1,58	1,62	1,65	1,69	1,72	1,76	1,80	1,84	1,87	1,91
$\epsilon_{cul,ck}$ , ‰	4,5	4,4	4,15	3,85	3,55	3,25	3,00	2,83	2,63	2,50	2,4
$\epsilon_{cul,cd}$ , ‰	3,75	3,70	3,59	3,44	3,28	3,10	2,93	2,72	2,57	2,43	2,29
$\epsilon_{cu3,cd}$ , ‰	3,38	3,33	3,23	3,10	3,00	2,8	2,64	2,45	2,31	2,19	2,06

**Table 2**

## Shear strength of concrete

Concrete strength class	C15	C20	C25	C30	C35	C45	C50	C55	C60
$C_{Rd,c}$ , MPa	0,18	0,22	0,26	0,30	0,34	0,37	0,41	0,44	0,48

**Table 3**

## Characteristic (normative) and calculated values of resistance and deformable characteristics of hot-rolled reinforcement by DSTU 3760-06

Characteristic	Grade of reinforcement			
	A240C	A400C	A500C	B 500
$f_{yk}$ , MPa	240	400	500	500
$f_{yd}$ , MPa	225	375(365)	435	435
$f_{ywd}$ , MPa	170	285	300	500
$E_s$ , MPa	$2,1 \times 10^5$	$2,1 \times 10^5$	$2,0 \times 10^5$	$1,9 \times 10^5$
$\epsilon_{s0}$	0,00107	0,00174	0,0021	0,0023
$\epsilon_{ud}$	0,025	0,025	0,02	0,02

**Table 4**Relation  $f_{yd}/f_{cd}$ 

Grade of concrete	Grade of reinforcement			
	A240C	A400C	A500C	Bp-I
C15	26,47	42,94	51,18	42,35
C20	19,56	31,74	37,82	31,30
C25	15,52	25,17	30,0	24,83
C30	13,24	21,47	25,58	21,77
C35	11,54	18,72	22,31	18,46
C40	10,23	16,59	19,77	16,36
C45	9,00	14,60	17,40	14,40
C50	12,86	13,27	15,82	13,09
C60	6,82	11,06	13,18	10,91

**Table 5**Limit values of the relative actual height of the compressed zone  $\xi_R$ 

Concrete		Reinforcement			
Grade	$\varepsilon_{cu3,cd}$ %	A240C	A400C	A500C	A500
		$\varepsilon_{yk} = 1,07\%$	$\varepsilon_{yk} = 1,74\%$	$\varepsilon_{yk} = 2,1\%$	$\varepsilon_{yk} = 2,3\%$
C10	3,38	0,769	0,660	0,617	0,595
C15	3,33	0,758	0,657	0,613	0,591
C20	7,23	0,751	0,650	0,606	0,584
C25	3,10	0,743	0,640	0,596	0,574
C30	3,00	0,737	0,633	0,588	0,566

**Table 6**

Coefficients for calculating rectangular sections

$\xi$	$\zeta$	$\alpha_m$	$\xi$	$\zeta$	$\alpha_m$	$\xi$	$\zeta$	$\alpha_m$
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
0,2	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	0,5	0,800	0,320	-	-	-

$$\alpha_m = 0,8\xi(1 - 0,4\xi); \zeta = 1 - 0,4\xi.$$

Table 6

**Рекомендовані мінімальні коефіцієнти поперечного армування**

Класи бетону за міцністю	Класи арматури		
	A240C	A400C	A500C
CI5 ... C25	0,0016	0,0009	0,0007
C30 ... C45	0,0024	0,0013	0,0011
C50 ... C60	0,0030	0,0016	0,0013

Table 7

Estimated cross-sectional areas and weight of reinforcement,  
(assortment of hot-rolled rod fittings according to DSTU 3760-16)

Diameter, mm	Design area of cross-section, cm <sup>2</sup> , with number of bars									Mass kg/m
	1	2	3	4	5	6	7	8	9	
6	0,283	0,57	0,85	1,13	1,41	1,70	1,98	2,26	2,54	0,222
8	0,503	1,01	1,51	2,01	2,51	3,02	3,52	4,02	4,52	0,395
10	0,785	1,57	2,36	3,14	3,93	4,71	5,50	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	7,70	9,24	10,78	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	12,57	15,71	18,85	21,99	25,13	28,27	2,466
22	3,801	7,60	11,40	15,21	19,01	22,81	26,61	30,41	34,21	2,984
25	4,909	9,82	14,73	19,63	24,54	29,45	34,36	39,27	44,18	3,853
28	6,157	12,31	18,47	24,63	30,79	36,94	43,10	49,26	55,42	4,834
32	8,042	16,08	24,13	32,17	40,21	48,25	56,30	64,34	72,38	6,313
36	10,18	20,36	30,54	40,71	50,89	61,07	71,25	81,43	91,61	7,990
40	12,57	25,13	37,70	50,26	62,83	75,40	87,96	100,53	113,10	9,865

Table 8

## Welded fabric assortment with transverse working armature

Type of welded fabric	Cross section of reinforcement per meter		Theoretical weight, kg
	Longitudal	Transversal	
$\frac{3Bp-I-250}{4Bp-I-200}$	0,28	0,63	0,71
$\frac{3Bp-I-250}{4Bp-I-150}$	0,28	0,84	0,81
$\frac{3Bp-I-250}{4Bp-I-100}$	0,28	1,26	1,21
$\frac{3Bp-I-250}{5Bp-I-200}$	0,28	0,98	0,99
$\frac{3Bp-I-250}{5Bp-I-150}$	0,28	1,31	1,24
$\frac{3Bp-I-250}{5Bp-I-100}$	0,28	1,96	1,75
$\frac{4Bp-I-250}{6A400C-200}$	0,28	1,41	1,32
$\frac{4Bp-I-250}{6A400C-150}$	0,5	1,89	1,88
$\frac{4Bp-I-250}{6A400C-100}$	0,5	2,83	2,61
$\frac{4Bp-I-250}{8A400C-200}$	0,5	2,51	2,59
$\frac{4Bp-I-250}{8A400C-150}$	0,5	3,36	3,03
$\frac{4Bp-I-250}{8A400C-100}$	0,5	5,03	4,34

**Table 9**Value of coefficients  $-\beta$ 

$\frac{V}{g}$	Point number										
	5	6	7	8	9	10	11	12	13	14	15
0,5	-0,0715	-0,01	+0,022	+0,024	-0,004	-0,0625	-0,003	+0,028	+0,028	-0,003	-0,0625
1,0	-0,0715	-0,02	+0,016	+0,009	-0,014	-0,0625	-0,013	+0,013	+0,013	-0,013	-0,0625
1,5	-0,0715	-0,026	-0,003	+0	-0,02	-0,0625	-0,019	+0,004	+0,004	-0,019	-0,0625
2,0	-0,0715	-0,03	-0,009	-0,006	-0,024	-0,0625	-0,023	-0,003	-0,003	-0,023	-0,0625
2,5	-0,0715	-0,033	-0,012	-0,009	-0,027	-0,0625	-0,025	-0,006	-0,006	-0,025	-0,0625
3,0	-0,0715	-0,035	-0,016	-0,014	-0,029	-0,0625	-0,028	-0,010	-0,010	-0,028	-0,0625
3,5	-0,0715	-0,037	-0,019	-0,017	-0,031	-0,0625	-0,029	-0,013	-0,013	-0,029	-0,0625
4,0	-0,0715	-0,038	-0,021	-0,018	-0,032	-0,0625	-0,030	-0,015	-0,015	-0,030	-0,0625
4,5	-0,0715	-0,039	-0,022	-0,02	-0,033	-0,0625	-0,032	-0,016	-0,016	-0,032	-0,0625
5,0	-0,0715	-0,04	-0,024	-0,021	-0,034	-0,0625	-0,033	-0,018	-0,018	-0,033	-0,0625

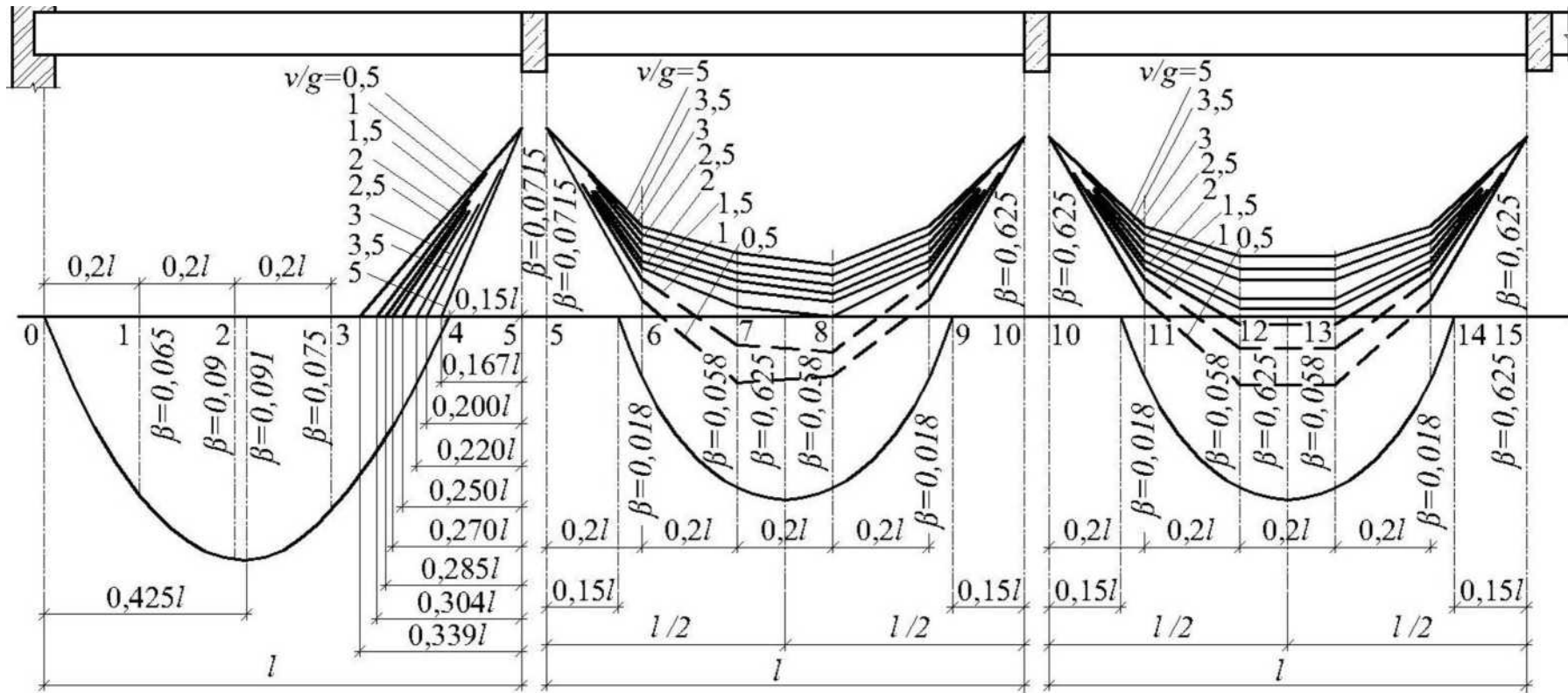


Fig. 1. Diagrams of bending moments for secondary beams

## REFERENCE

1. ДБН В.2.6-98:2009. Конструкції будинків і споруд. Бетонні та залізобетонні конструкції. Основні положення. УКНД 91.080.40 [Текст] : затв. : Наказ Мінрегіонбуду України 24.12.09. № 680 / Мінрегіонбуд України. - К., 2011. - 71 с.
2. ДСТУ Б В.2.6-156:2010. Конструкції будинків і споруд. Бетонні та залізобетонні конструкції з важкого бетону. Правила проектування. УКНД 91.080.40 [Текст] : затв. : Наказ Мінрегіонбуду України 28.12.10. № 566 / Мінрегіонбуд України. - К., 2011. - 118 с.
3. ДСТУ 3760: 2006. Прокат арматурний для залізобетонних конструкцій. Загальні технічні умови. УКНД 77.140.15 [Текст] : затв. : Наказ Держспоживстандарту України 11.12.06. № 343 / Держспоживстандарт України. - К., 2007. - 28 с.
4. EN 1992-1-1-2004: Eurocod 2: Design of concrete structures. - Part 1-1: General rules and rules for buildings.
5. Мурашко Л.А. Розрахунок за міцністю перерізів, нормальних до поздовжньої осі згинальних залізобетонних елементів за ДБН В.2.6-98:2009. Навч. посіб / Мурашко Л.А., Колякова В.М., Сморгалов Д.В. - К. :КНУБА, 2012. - 62 с.
6. Железобетонные конструкции: Курсовое и дипломное проектирование / Под ред. А.Я. Барашикова. - К. : Вища шк., 1987. - 416 с.
7. Проектирование железобетонных конструкций: Справоч. пособ / Под ред. А. Б. Гольшева. - К. : Будівельник, 1990. - 542 с.