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 2
“Design of slabs prefabricated elements,
columns and foundations 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”)

Kharkiv
O. M. Beketov NUUE
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Metodological guidelines for execution of design project № 1 in the discipline “Reinforced-Concrete and Masonry Structures” Section 2 “Design of slabs prefabricated elements, columns and foundations 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”) / O. M. Beketov National University of Urban Economy in Kharkiv ; com. : P. A Reznik, N. O. Psurtseva. – Kharkiv : O. M. Beketov NUUE, 2021. – 41 p.

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N. O. Psurtseva

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

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CONTENTS

GENERAL GUIDELINES

These recommended practices are the second section for the implementation of the course number 1 on the discipline “Reinforced concrete and Stone structures”.

The recommendations as to the layout of slabs and reference points remain the same as in the first section of the project, which deals with the design of a monolithic reinforced concrete slab and girder floor.

A prefabricated option allows another arrangement (the direction of the girds does not necessarily coincide with the direction of the main beams, the bays of the girds and the slabs can be different from the bays of the main and secondary beams).

Students are invited to develop two options of buildings:

industrial and civil (according to the task).

To facilitate the students work over the course project, the both options of calculations of all elements of floor slabs and frameworks are considered at practical classes.

1 SLAB’S ARRANGEMENT

The floor slab is made up of prefabricated slabs, which are laid on the gird.

Subject to the number of bays, the girds of extreme bays (B1) and middle bays (B2) differ from each other both in the appearance (formwork) and the reinforcement pattern.

Subject to the building option (industrial or civil), the floor slab is arranged in different ways.

Recommendations are provided for each type of building.

A) Civil building.

The building girds have a T-section with shelves in the lower area.

The gird has an “ndercutting” in the column support, and a rectangular profile (fig. 1.1).

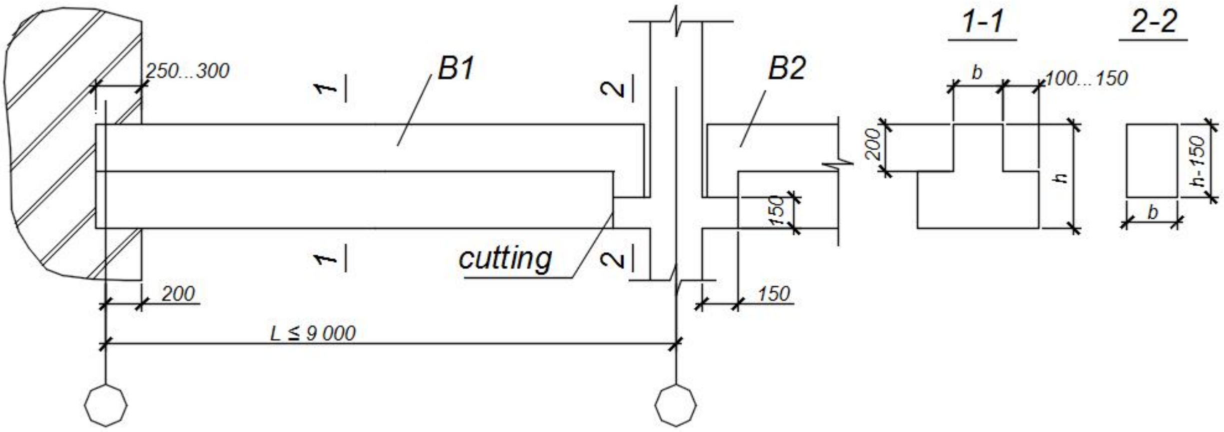


Figure 1.1 – Civil building beam

The gird's height is assumed as $h = (1/10...1/12)L$, profile width $b = (0,3...0,5)h$.

Actual cross-sectional dimensions are assumed as multiple of 50 mm.

The floor slabs (panels) have a standard nominal width of 800...1 400, 1 500, 1 600...2 000 mm (in 200 mm).

The height of the slabs is standard – 220 mm.

The slabs are made with round cavities which are 159 mm in diameter (fig. 1.2).

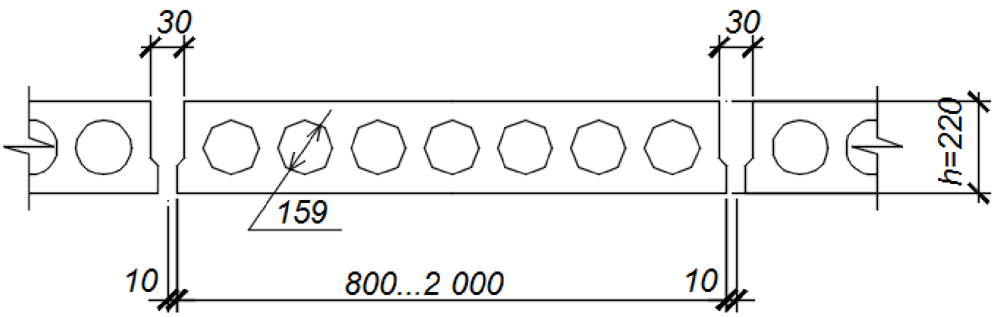


Figure 1.2 – Slabs with round cavities

The slabs are placed on the girds' shelfs (in the first bay – on the wall).

The choice of slabs nominal sizes is assumed to cover the width from the inner surface of the wall to the edge of the column or between the columns faces.

The remaining lengths that are not overlapped with slabs, are to be filled with monolithic reinforced concrete.

The dimensions of the columns cross section are standard – 300 mm ×300 mm (for a significant load – 350 mm ×350 mm).

Б) Industrial building

The girds have a complex profile (fig. 1.3).

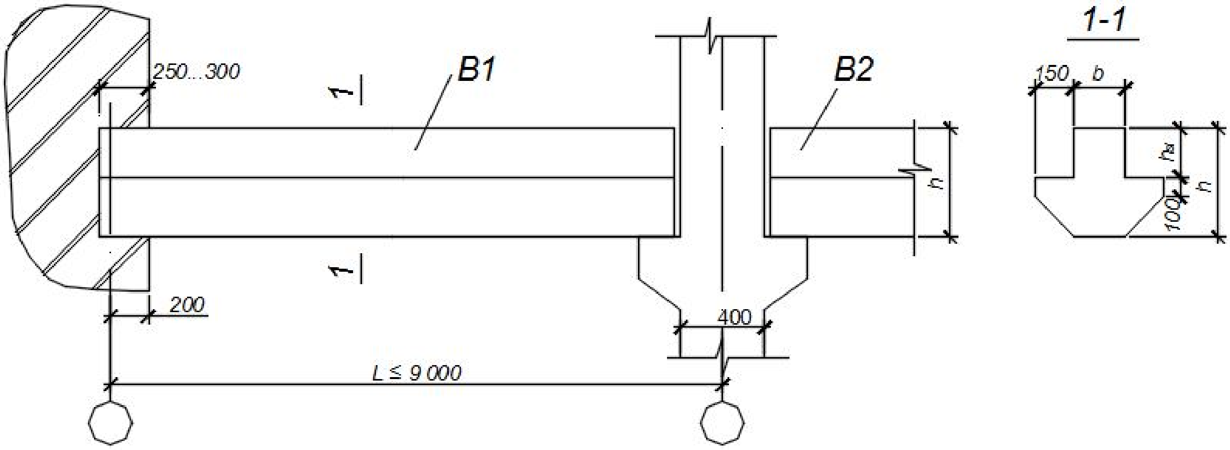


Figure 1.3 – Industrial building beam

Dimensions of the gird's cross section $h = (1/10\ldots1/12)L$, $b = (0{,}3\ldots0{,}5)h$.

Actual cross-sectional dimensions are assumed as multiple of 50 mm.

The floor slabs have a П-shaped profile and can have nominal width of 1200 mm and 1500 mm (fig. 1.4).

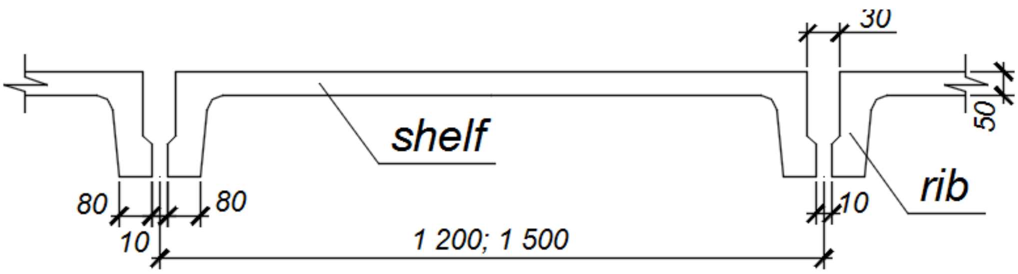


Figure 1.4 – Industrial building slabs

The arrangement of the floor slab is executed to cover the distance between the inner surfaces of the opposite walls.

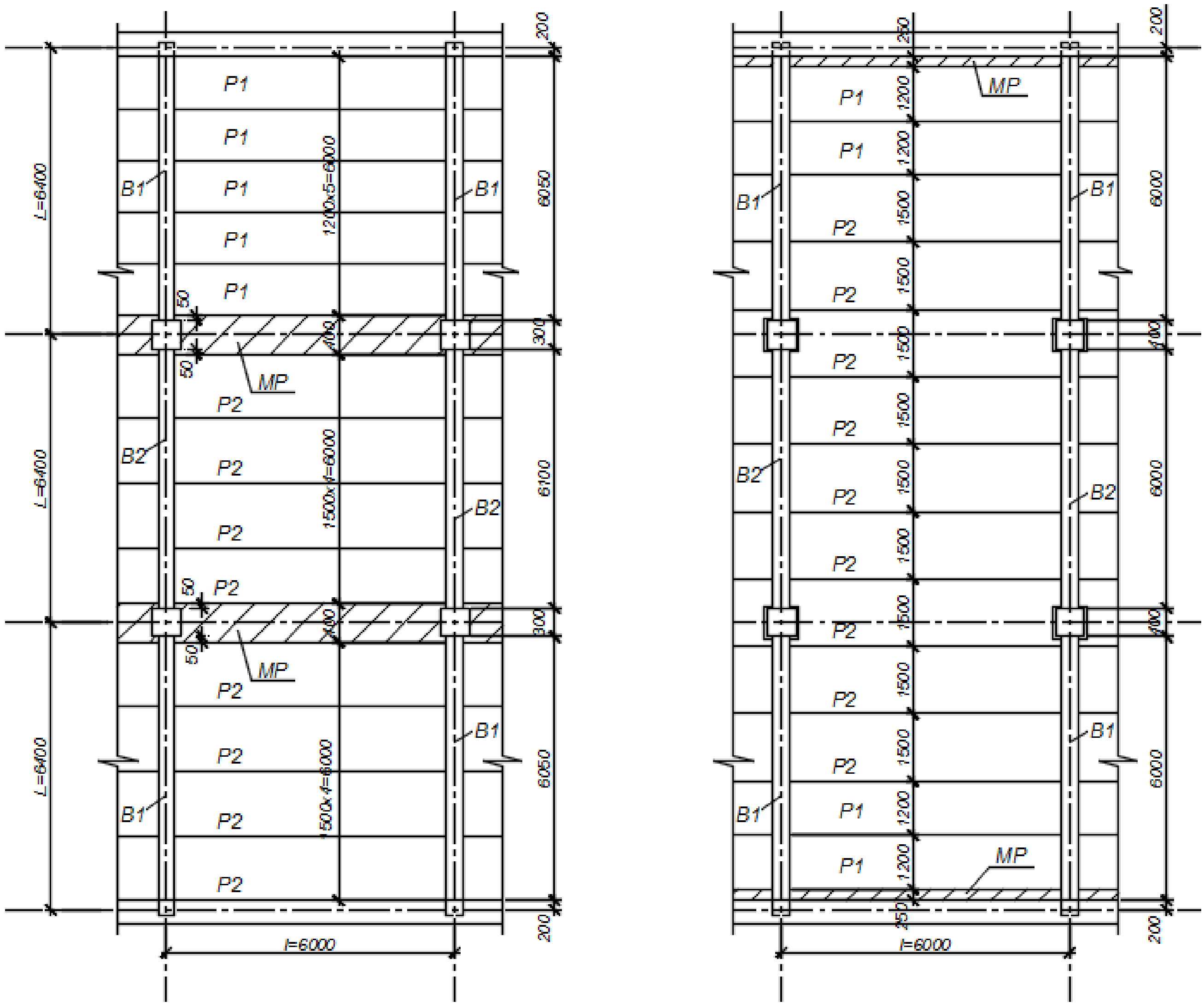
In this case, the slab arrises are supported by the gird's shelves and cannot coincide with the columns (the columns cross section – 400 mm ×400 mm).

Therefore, the slabs arrangement is recommended to start from the middle, from the columns, so that the edges of one slab could be supported by the girds of adjacent bays.

The shelf of this slab has a cutout that covers the column.

The remaining width of the building which is not covered by the slabs is subject to a monolithic concreting. Normally, this remaining width is concreted near the wall.

Examples of the possible layout of the floor slabs in both types of buildings are shown on the fragments (fig. 1.5).



a) b)

Figure 1.5 – Examples of the possible layout of the floor slabs:
a – Civil building; b – Industrial building

2 AN EXAMPLE OF A CIVIL BUILDING FLOOR SLAB ELEMENTS CALCULATION

2.1 Loading per 1 m^2 of the floor slab

Table 2.1 – Collecting the load

№ in ord.	Load type	Characteristic load value, kN/m^2	Reliability index γ_f	Design load value, kN/m^2
	A) Constant			
1	*Parquet* (δ = 2 cm, ρ = 6 kN/m^3) 0,02·6	0,12	1,2	0,144
2	Cement slurry (δ = 3 cm, ρ = 20 kN/m^3) 0,03·20	0,6	1,3	0,78
3	Sound insulation – foam concrete (δ = 6 cm, ρ = 7 kN/m^3) 0,06·7	0,42	1,3	0,546
4	Reinforced concrete slab (2,7…3,1 kN/м^2)	3,0	1,1	3,3
	Total constant			g = 4,77
5	*B) Variable (as per the task)*	3,5	2	v = 4,2
	C) Total			$q = g + v \approx$ ≈ 9,0

2.2 Static calculation of the plate with a nominal width b_{pl} = 1,5 m

The calculation effort is defined as for a single-span loosely supported beam.

Pre-assume the dimensions of the gird cross-section: h = 60 cm, b = 25 cm, width of the gird's shelves – 10 cm. The width of the slab's resting against the gird's shelf c = 9 cm. The design bay of the slab is equal to the distance from the centers of slab's resting against the girds' shelves (fig. 2.1):

$l_0 = l - b - c - 2$ gaps = 6 000 – 250 – 90 – 20 = 5 640 mm = 5,64 m.

Line loading $q = q_{1\text{m}^2} \cdot b_{pl} = 9 \cdot 1{,}5 = 13{,}5$ kN/m.

Design effort

$M_{max} = ql^2_0/8 = 13{,}5 \cdot 5{,}64^2/8 = 53{,}7$ kNm;

$V_{max} = ql_0/2 = 13{,}5 \cdot 5{,}64/2 = 38{,}1$ kN.

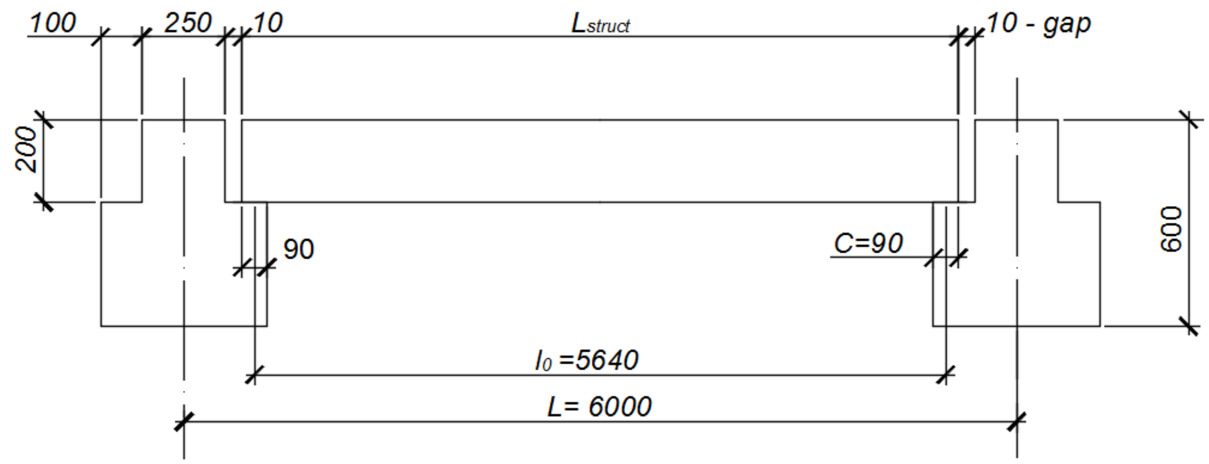


Figure 2.1 – Determination of the calculated span of the plate

2.3 The slab's design calculation

Concrete of grades C12/15, C16/20, C20/25 is accepted for the slab manufacture.

Depending on the grade, the calculated characteristics of the concrete strength are accepted (Appendix A).

For the example under consideration, we accept concrete of grade C16/20 with estimated strengths f_{cd} =11,5 MPa for compression and f_{ctd} = 0,87 MPa for stretching.

The working reinforcement is assumed as grade A400C with the design strength of f_{yd} = 365 MPa (Appendix C).

The cross section of the slab has 7 round cavities and is considered as a double-T shape (fig. 2.2).

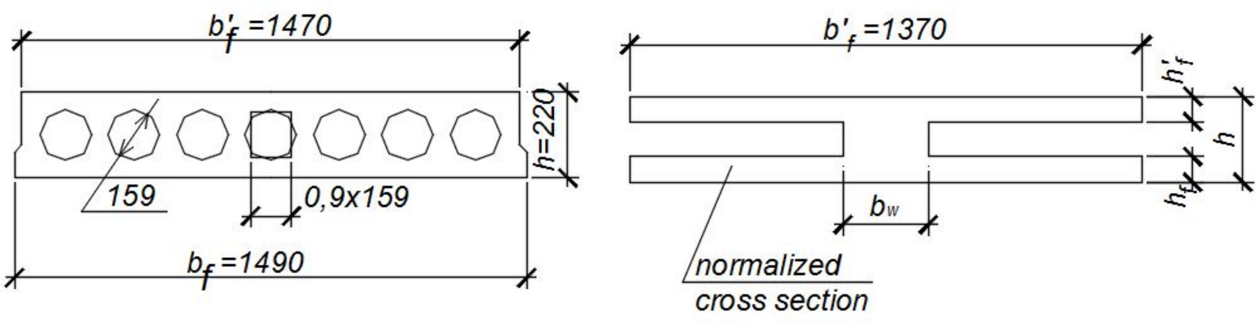


Figure 2.2 – Calculated section of the plate

We assume the following cross section dimensions for calculation:

b_{eff} = 147 cm; h = 22 cm; b_w = 147 – 7·0,9·15,9 = 46,8 cm; d = 19,5 cm; $h_f = h'_f$ = (22 – 0,9·15,9)/2 = 3,845 cm.

The position of the neutral layer is to be determined by the magnitude of the moment M_f :

$$M_f = f_{cd}\, b_{eff}\, h'_f(d - 0{,}5\, h'_f) = 1{,}15\cdot147\cdot3{,}845(19{,}5 - 0{,}5\cdot3{,}845) =$$

$$= 11486 \text{ kNcm} = 114{,}86 \text{ kNm} > M = 53{,}7 \text{ kNm}.$$

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

$\alpha_m = M_{max}/f_{cd}b_{eff}d^2$ = 5370/1,15·147·19,5^2 = 0,103; ζ = 0,945 (Appendix D);

$A_s = M_{max}/\zeta f_{yd} d$= 5370/0,945·36,5·19,5 = 7,98 cm^2.

The working reinforcement is located along the edges of the cross section and between the cavities.

We assume the working longitudinal reinforcement as 8Ø12A400C (As = 9,05 cm^2, Appendix E).

This fitting is used as a bottom grid G1. where the transverse structural reinforcement is assumed as Ø3B500 with a step of 250 mm.

Then we examine the need to calculate the strength of sloping cross sections.

The concrete carrying capacity

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

where $C_{Rd,c}$ = 0,138 5; $\rho_1 = A_s/b_w d$ = 9,05/46,8·19,5 = 0,009 9;

$$K = 1+\sqrt{\frac{200}{d}} = 1+\sqrt{\frac{200}{195}} = 2{,}01 > 2;\quad \text{assume } K = 2.$$

$$V_{Rd,c} = (0{,}138\,5\times2\sqrt[3]{100\cdot 0{,}0099\cdot 15}\,)468\cdot195 = 62\,135\text{ H} = 62{,}13\text{ kN} > V_{max} = 38{,}1\text{ kN}.$$

Crosswise reinforcement is to be assumed structurally – Ø3B500 with a step of 100 mm.

The rebar frameworks with crosswise reinforcement (Fr1) is to be assumed as 1/4l = 1500 mm long and placed in supporting areas by 4 frameworks in per cross section (fig. 2.3). In the slab's upper area, we place a grid G2 (Ø3B500 with a cell 200 mm x 200 mm) structurally.

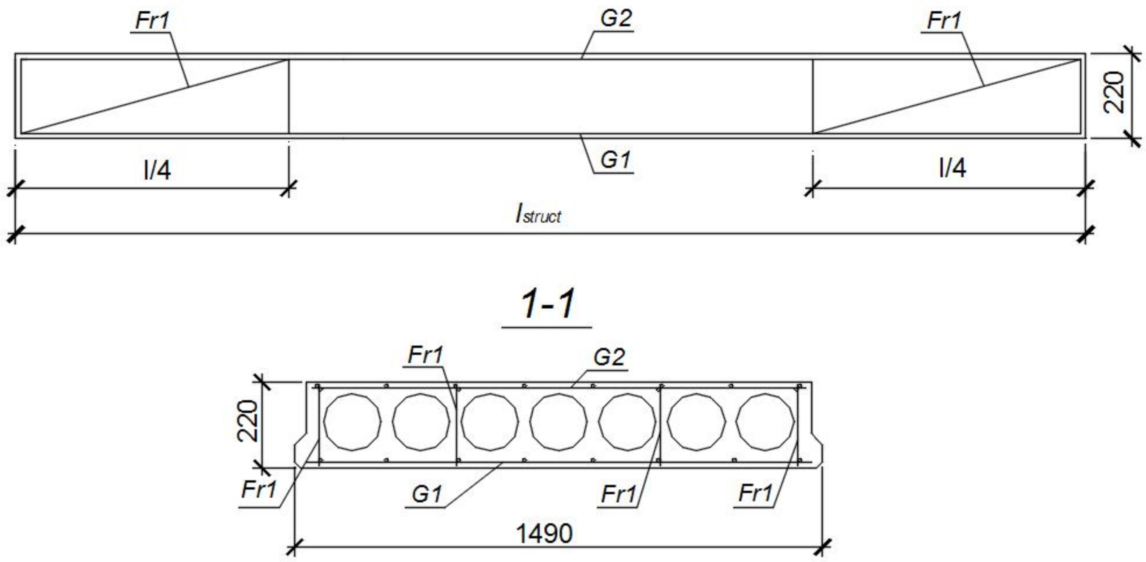


Figure 2.3 – Reinforcing plates with round cavities

2.4 The final bay bent. Static calculation

The design bay of the final bay gird is assumed as the distance from the middle of its reliance on the wall to the central axis of the column (fig. 2.4).

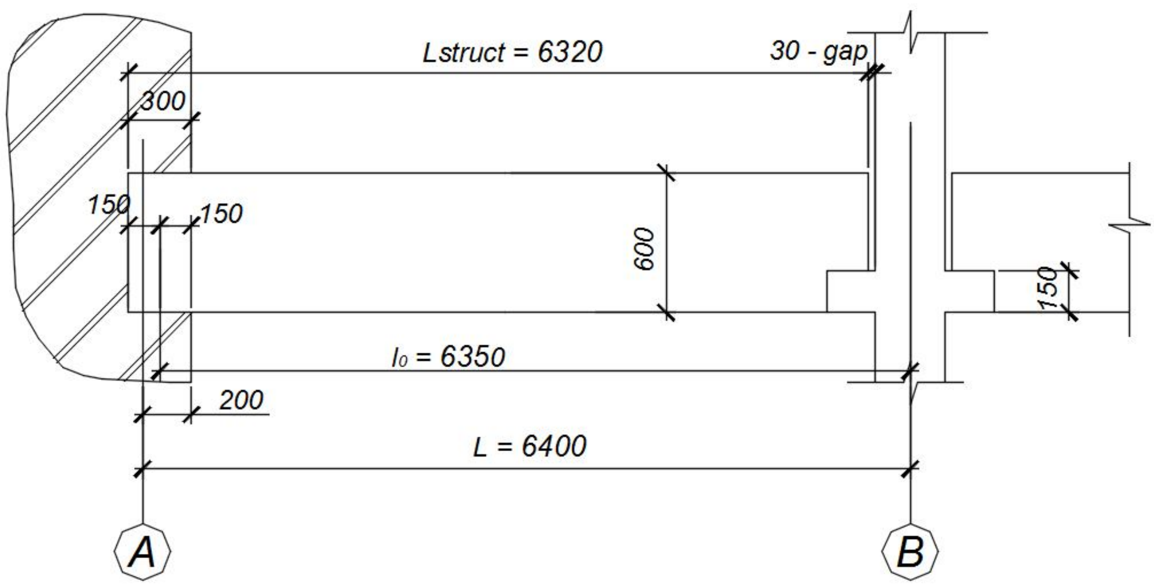


Figure 2.4 – Estimated spindle beam

The mounting resting against the wall is considered to be a hinged one, and the reliance on the column – a rigid one due to welding of the gird's and column's embedded parts.

The gird's design bay l_0 = 6 400–200+150 = 6 350 mm = 6,35 m.

The gird's design length

$L_{struct.} = l_0$+150 mm – h_c/2 – gap = 6 350+150–300/2–30 = 6 320 мм.

– constant $g = g_{1м2}\cdot l + \gamma_f$ (individual weight of the gird's lower part with shelves) = 4,77·6 +1,1(0,6 – 0,2)0,45·25 = 33,6 kN/m;

– variable $v = v_{1м2}\cdot l$ = 4,2·6 = 25,2 kN/m;

– total $q = g + v$ = 33,6 + 25,2 = 58,8 kN/m.

To calculate the final bay, two calculation schemes are considered:

– the constant load of all bays of the gird with variable load of the final bay and further through the bay;

– the constant load of all bays of the gird with the variable load of the first and second bays.

The largest bending moment is determined from the first scheme (Attachment 6)

$M_1 = (0{,}08g + 0{,}101v)l_{01}^2$ = (0,08·33,6 + 0,101·25,2)6,352 = 211 kNm.

The largest moment at support is determined from the second scheme (Appendix 6)

$M_B = (–0{,}1g – 0{,}117v)l_{01}^2$ = (–0,1×33,6 –0,117·25,2)6,352 = – 254,4 kNm.

Due to redistributing efforts one can reduce the moments in the bay and on the support: $M'_1 = 0{,}9M_1$ = 0,9·211 = 189,9 kNm;

$M'_B = /0{,}75M_B/$ = 0,75·254,4 = 190,8 kNm.

Transverse forces

$V_A = (0{,}4g+0{,}45v)l_{01}$ = (0,4·33,6+0,45·25,2)6,35 = 157,4 kN;

$V_B = (–0{,}6g - 0{,}617v)l_{01}$ = (–0,6·33,6 –0,617·25,2)6,35 = –226,8 kN.

For a design calculation, the following is finally assumed

– in the span $M_{sp} = M'_1$ = 189,9 kNm;

– on supports $M_{sup} = M'_B - |\ 0{,}5 V_B\ h_к\ | = 190{,}8 - 226{,}8 \cdot 0{,}15 =$ $= 156{,}8$ kNm.

2.5 Calculation of the gird strength in normal cross sections

For the considered example, we accept concrete of grade C20/25 (f_{cd} = 14,5 MPa, f_{ctd} = 1,0 MPa), working rebar – grade A400C (f_{yd} = 365 MPa).

A) Rebar in the bay

The cross section's working height is assumed as d = 55 cm.

$$\alpha_m = M_{sp}/ f_{cd} b d^2 = 18\ 990/1{,}45 \cdot 25 \cdot 55^2 = 0{,}173; \qquad \zeta = 0{,}904;$$

$$A_s = M_{sp} / \zeta f_{yd}\, d = 18\ 990/0{,}904 \cdot 36{,}5 \cdot 55 = 10{,}46 \text{ cm}^2.$$

Assume 2Ø20A400C + 2Ø18A400C (A_s = 11,37 cm²).

B) Rebar on the support (cross section's working height d = 41cm)

$$\alpha_m = M_{sup}/ f_{cd}\ b\ d^{\ 2} = 15\ 680/1{,}45 \cdot 25 \cdot 41^2 = 0{,}257; \quad \zeta = 0{,}85;$$

$$A_s = 15\ 680/0{,}85 \cdot 36{,}5 \cdot 41 = 12{,}32 \text{ cm}^2.$$

Assume 2Ø28A400C (A_s = 12,32 cm²).

2.6 Calculation of the gird strength in sloping sections

The calculation is performed individually for the transverse force V_A = 157,4 kN for the full cross section with h = 60 cm and for the transverse force V_B = 226,8 kN, for the cross section with the 'undercutting' on the support *B* (*h* = 45 cm).

As an example, the calculation is based on $V_{max} = V_B$ = 226,8 kN.

Calculation is performed in the following order.

1. We assume the crosswise reinforcement in two frameworks under condition of welding 2Ø10A400C (As = 1,57 cm2); collar clamps' step $s_w \leq 0{,}75d = 0{,}75 \cdot 41 = 30{,}75$ cm.

2. We determine the concrete carrying capacity

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

where $C_{Rd,c} = 0{,}1385$; $K = 1 + \sqrt{200/d} = 1 + \sqrt{200/410} = 1{,}7 < 2$;

ρ_1= 12,32/25×41= 0,012;

$$V_{Rd,c\ 1} = (0{,}1385 \cdot 1{,}7\sqrt[3]{100 \cdot 0{,}012 \cdot 18{,}5}) = 67828 \text{ H} = 67{,}83 \text{ kN};$$

$$V_{Rd,c\ 2} = (0{,}035\sqrt{f_{ck}K^3})b_w d = (0{,}035\sqrt{18{,}5 \cdot 1{,}7^3})250 \cdot 410 = 34202 \text{ H} = 34{,}2 \text{ kN}.$$

Assume $V_{Rd,c}$ = 67,83 kN < V_{Ed} = 226,8 kN.

Crosswise reinforcement is required based on calculation.

The collar clamps carrying capacity

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

where $z = 0{,}9d = 0{,}9 \cdot 41 = 36{,}9$ cm; value $V_{Ed}/b_w d = 226\ 800 / 250 \cdot 410 = 2{,}21$; as per the schedule (fig. 2.5) $ctg\theta = 2{,}5$; $tg\theta = 0{,}4$.

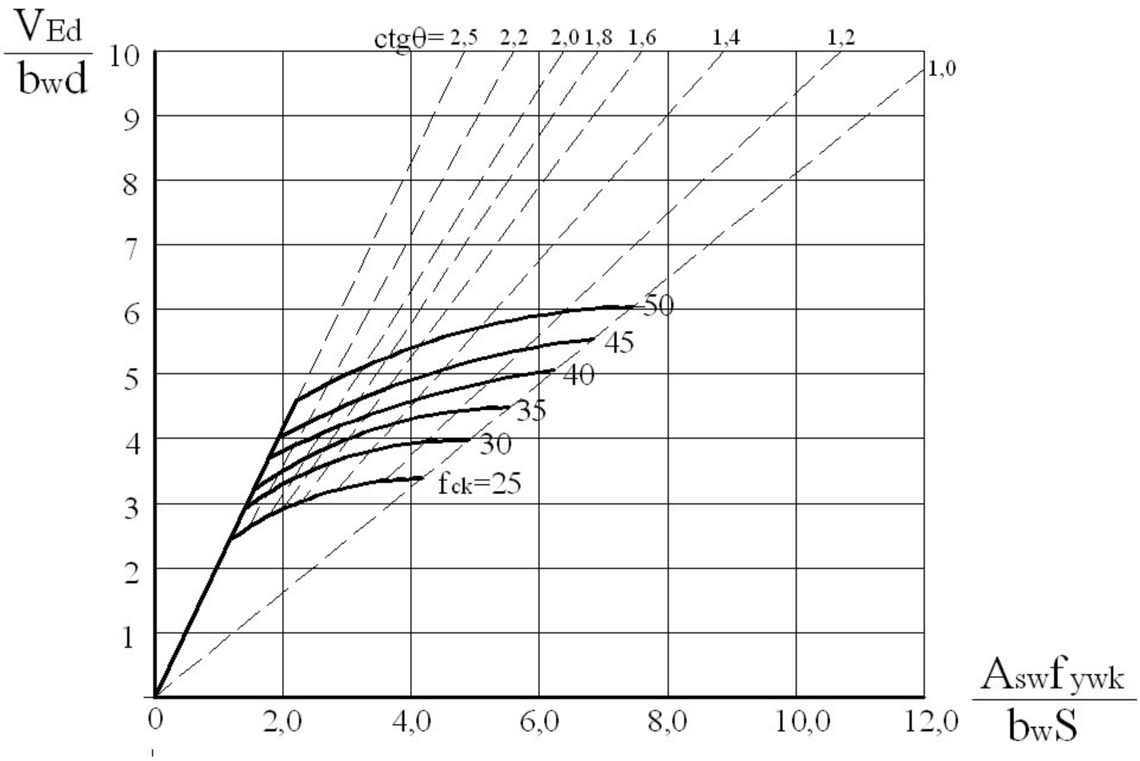


Figure 2.5 – Schedule for determining

$$V_{Rd,s} = 1{,}57 \cdot 36{,}9 \cdot 29 \cdot 2{,}5 / 25 = 168 \text{ kN}.$$

$$V_{Rd\ max} = \alpha_{cw} b_w z v_1 f_{cd} / (ctg\theta + tg\theta),$$

where $\alpha_{cw} = 1$ when there is no prestressing; $v_1 = 0{,}6$ for $f_{ck} \leq 60$ MPa;

$$V_{Rd\ max} = 1 \cdot 25 \cdot 36{,}9 \cdot 0{,}6 \cdot 1{,}45/(2{,}5 + 0{,}4) = 276{,}7 \text{ kN}.$$

We assume the lesser value as $V_{Rd,s}$ = 168 kN.

The cross section full carrying capacity

$V_{Rd} = V_{Rd,c} + V_{Rd,s} = 67,83 + 168 = 230,54$ kN > $V_{Ed} = 226,8$ kN..

The cross section carrying capacity is provided.

On *A* support, we pre-assume in the two frameworks 2Ø8A400C, the step in the support-adjacent area (0,25*l*) s_{w1} = 200 mm. The step of the crosswise reinforcement in the middle area of the gird's length is to be assumed as s_{w2} = 400 mm ($s_{w2} \leq 0,75h$ = 450 mm).

All further checks are performed in complianc with the previously provided algorithm.

2.7 A civil building gird design

The bay-based working reinforcement is to be arranged in two frameworks Fr1. The upper reinforcement in these frameworks is assumed structurally (2Ø12A240C).

The supporting working reinforcement is arranged in two frameworks Fr2.

The length of frameworks Fr2 – a quarter of the gird's bay.

The lower reinforcement of frameworks Fr2 is assumed structurally.

In most designs it is assumed as the diameter of the lower bay reinforcement (2Ø18A400S). Reinforcing the shelves of the gird is performed with bent frameworks Fr3. The gird reinforcement scheme is shown in figure 2.6.

3 AN EXAMPLE OF AN INDUSTRIAL BUILDING SLAB ELEMENTS CALCULATION

3.1 Loading per a floor slab 1 m²

Table 3.1 – Collecting the load

№ in ord.	Load type	Characteristic load value, kN/m²	Reliability index γ_f	Design load value, kN/m²
1	2	3	4	5
1	*A) Constant* Inlay covering (δ=2 cm, ρ=20 kN/m³) 0,02·20	0,4	1,3	0,52
2	Sound insulation (foam concrete) 0,06·8	0,48	1,3	0,624

Continuation of the talica 3.1

1	2	3	4	5
3	Reinforced concrete slab (1,8…2,5 kN/m^2)	2,4	1,1	2,64
4	Total constant *B) Variable* (as per the task)	8,0	1,2	g = 3,784 v = 9,6
	Total		$q = g + v = 13{,}39$	

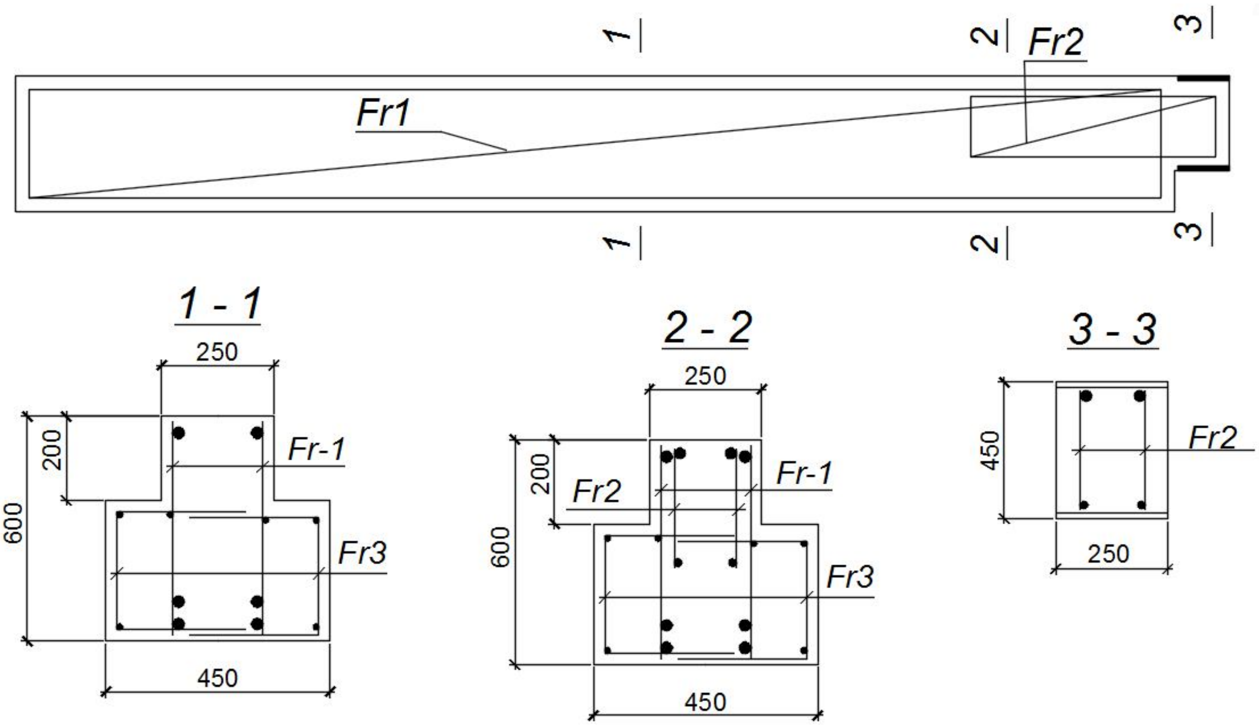


Figure 3.1 – Scheme of beam reinforcement

3.2 Static calculation of the slab with nominal width b_{pl} = 1,5 m

The dimensions of the gird's cross-section are to be assumed as:

h = 65 cm, b = 25 cm, the width of the supporting shelves 15 cm, the slab's height is assumed as h_{pl} = 35 cm.

The slab’s design bay is to be determined in accordance with figure 3.2:

l_0 = 6 000 – 250 – 30·2 – 120 = 5 570 mm = 5,57 m.

Line load $q = q_{1m^2} \cdot b_{pl}$ = 13,39·1,5 = 20,1 kN/м.

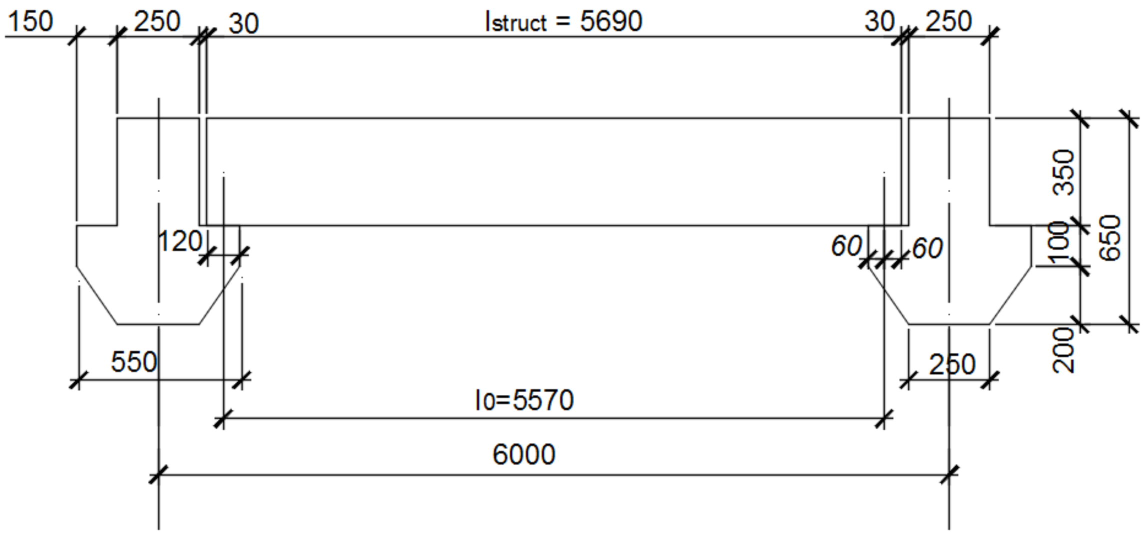


Figure 3.2 – To the definition of the calculated span of the plate

Design efforts $M_{max} = ql_0^2/8 = 20{,}1 \cdot 5{,}57^2 \,/\, 8 = 78$ kNm,

$$V_{max} = ql_0/2 = 20{,}1 \cdot 5{,}57 \,/\, 2 = 56 \text{ kN}.$$

3.3 Design calculation of the slab

We assume the concrete grade as C16/20 (f_{cd} = 11,5 MPa), working reinforcement – grade A400C (f_{yd} = 365 MPa).

The cross section of П-shaped slab is considered as T-shaped with a shelf in the compressed area.

The slab's cross section is given in figure 3.2.

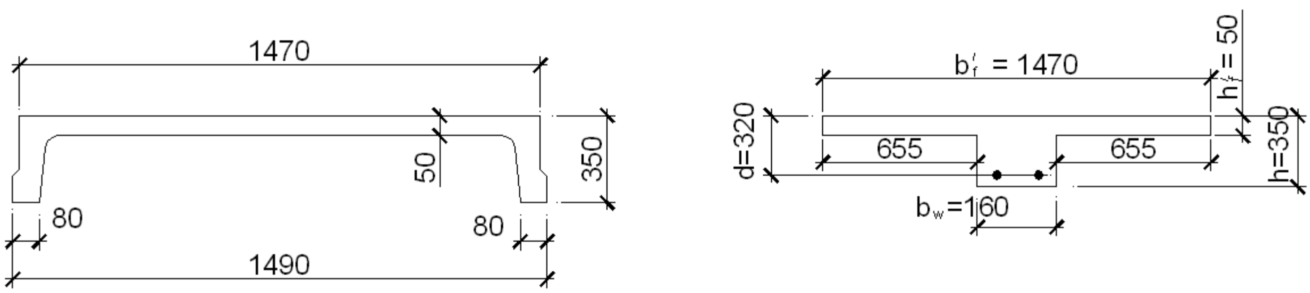


Figure 3.3 – Calculated section of the plate

$b_{eff} = 2(0{,}2b_1 + 0{,}1l_0) + b_w = 2(0{,}2 \cdot 65{,}5 + 0{,}1 \cdot 557) + 16 = 153{,}6$ cm $> b_{факт.}$

Assume b_{eff} = 147 cm.

The neutral layer location

$$M_f = f_{cd}\, b_{eff}\, h_f'(d\ - 0.5h_f') = 1{,}15 \cdot 147 \cdot 5(32 - 2{,}5) = 24\,934 \text{ kNcm} =$$

$$= 249{,}3 \text{ kNm} > M_{max} = 78 \text{ kNm}.$$

The neutral layer is located within the compressed shelf, so the cross section is calculated as rectangular.

$$\alpha_m = 7\,800/1{,}15\cdot147\cdot32^2 = 0{,}045; \quad \zeta = 0{,}977;$$

$$A_s = 7\,800\,/\,0{,}977\cdot36{,}5\cdot32 = 6{,}84\ \text{cm}^2.$$

The working reinforcement is to be assumed as 2Ø22A400C (As = 7,6 cm^2) and placed in two frameworks (one framework per the slab arris).

We assume the upper reinforcement in these frameworks structurally as 2Ø12A240C.

We assume crosswise reinforcement under condition of welding with the working reinforcement (Ø6A240C). Collar clamps' step is $s_w \le 0{,}75d = 0{,}75\cdot32 =$ 24 см. Assume $s_w = 20$ cm.

The concrete carrying capacity

$\rho_1 = A_s/b_wd = 7{,}6/16\cdot32 = 0{,}0148.$

$K = 1 + \sqrt{\dfrac{200}{d}} = 1 + \sqrt{\dfrac{200}{320}} = 1{,}79 < 2.$

$V_{Rd,c\,1} = (0{,}138\ 5\cdot1{,}79\sqrt[3]{100\cdot0{,}0148\cdot15})160\cdot320 = 35674$ H $= 35{,}67$ kN;

$V_{Rd,c\,2} = (0{,}035\sqrt{15\cdot1{,}79^3})160\cdot320 = 16600$ H $= 16{,}6$ kN.

Assume $V_{Rd,c} = 35{,}67$ kN $< V_{Ed} = 56$ kN.

Collar clamps are required based on calculation.

The collar clamps carrying capacity

$V_{Rd,s} = A_{sw}zf_{ywd}ctg\theta/s_w$; $\quad z = 0{,}9d = 0{,}9\cdot32 = 28{,}8$ cm

For value $V_{Ed}/b_wd = 56\,000/160\cdot320 = 1{,}093$ N/mm^2; $\quad$ ctgθ = 2,5.

$V_{Rd,s} = 0{,}57\cdot28{,}8\cdot17\cdot2{,}5\,/\,20 = 34{,}88$ kN

The cross section full carrying capacity

$$V_{Rd} = V_{Rd,c} + V_{Rd,s} = 35{,}67 + 34{,}88 = 70{,}55\ \text{kN} > V_{Ed} = 56\ \text{kN}.$$

The cross section carrying capacity is provided.

The slab's shelf should be reinforced with a grid G1 – Ø4B500 with mesh 200 mm × 200 mm (fig. 3.4).

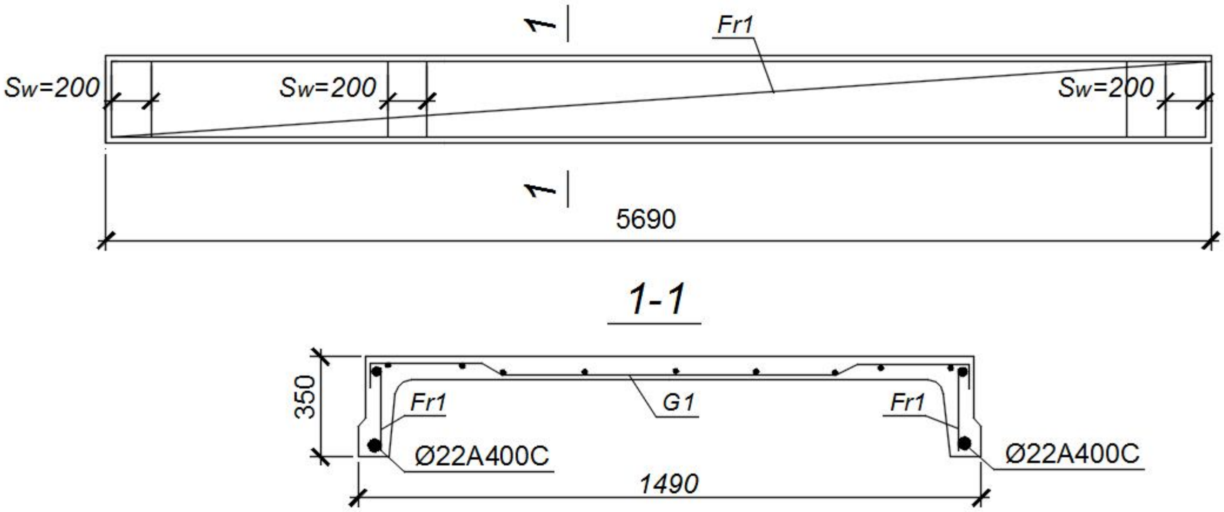


Figure 3.4 – Reinforcing the ribbed slab

3.4 Final flight bolt. Static calculation

The design bay is assumed as the grid for a civil structure (l_0 = 6,35 m).

The grid's design length is equal to (fig. 3.5).

$l_{stryctr.} = l_{01}$ + 150 mm – $h_{col}/2$ – gap = 6350 + 150 – 200 – 50 = 6250 mm.

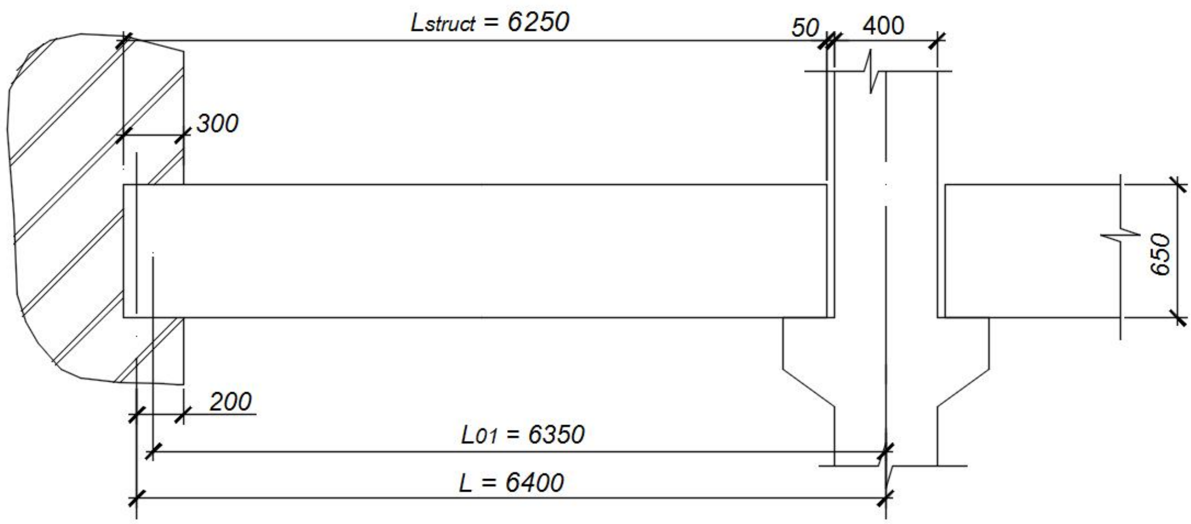


Figure 3.5 – To determine the calculated length of the beam

Design line load

– constant $g = g_{1m^2} l + \gamma_f \cdot$(individual weight of the grid's lower part) =

= 3,784·6 + 1,1(0,55·0,35 – 0,15·0,2)25 = 27,2 kN/m;

– variable $v = v_{1m^2} l$ = 9,6·6 = 57,6 kN/m;

– full $q = g + v$ = 27,2 + 57,6 = 84,8 kN/m.

To calculate the final bay, two calculation schemes are considered (see p. 2.4), from which the bending moments and transverse forces are determined:

$$M_1 = (0{,}08 \cdot 27{,}2 + 0{,}101 \cdot 57{,}6)\ 6{,}35^2 = 322{,}3 \text{ kNm};$$

$$M_B = (-0{,}1 \cdot 27{,}2 - 0{,}117 \cdot 57{,}6)6{,}35^2 = -\ 381{,}4 \text{ kNm};$$

$$V_A = (0{,}4 \cdot 27{,}2 + 0{,}45 \cdot 57{,}6)6{,}35 = 233{,}7 \text{ kN};$$

$$V_B = (-0{,}6 \cdot 27{,}2 - 0{,}617 \cdot 57{,}6)6{,}35 = -\ 329{,}3 \text{ kN}.$$

Due to the redistribution of efforts, the design bending moments in the bay and on the support

$$M_{sp} = 0{,}9\ M_1 = 0{,}9 \cdot 322{,}3 = 290 \text{ kNm};$$

$$M_{sup} = /0{,}75 M_B\ / - /0{,}5 V_B h_{col}/ = 0{,}75 \cdot 381{,}4 - 329{,}3 \cdot 0{,}4/2 = 220 \text{ kNm}.$$

3.5 Calculation of the gird strength in normal cross sections

We assume the concrete as C20/25 grade, and the rebar of A400C

A) Reinforcement in the bay

The working height of the cross section is d = 59 cm for the reinforcement double-row arrangement.

$$\alpha_m = 29\ 000/1{,}45 \cdot 25 \cdot 59^2 = 0{,}23; \qquad \zeta = 0{,}867;$$

$$A_s = 29\ 000\ /\ 0{,}867 \cdot 36{,}5 \cdot 59 = 15{,}53 \text{ cm}^2.$$

Assume 2∅25A400C + 2∅22A400C (A_s = 17,42 cm^2).

*B) Reinforcement on the support (*for single-row spacing of the reinforcement d = 61 cm)

$$\alpha_m = 22\ 000/1{,}45 \cdot 25 \cdot 61^2 = 0{,}163; \quad \zeta = 0{,}91;$$

$$A_s = 22\ 000\ /0{,}91 \cdot 36{,}5 \cdot 61 = 10{,}86 \text{ cm}^2.$$

Assume 2∅28A400C (A_s = 12,32 cm^2).

3.6 Calculation of the gird strength in sloping sections

The calculation is performed on $V_{Ed} = V_{max} = V_B$ = 329,3 kN.

The crosswise reinforcement is assumed in two frameworks 2∅8A400C (A_s = 1,01 см^2).

The crosswise reinforcement step is $s_w \leq 0{,}75d$ = 45 см. We assume in the extreme bay quarters as sw1 = 20 cm, and in the mid part of the bay as s_{w2} = 40 cm.

Verification of the durability of sloping cross sections is to be carried out using the above algorithm.

We believe that all inspections have been carried out and the strength of sloping cross sections with the accepted reinforcement is sufficient.

3.7 The gird designing. The Lean Reinforcement

The bay working reinforcement should be located in two Kp1 frameworks.

The upper reinforcement in these frameworks is to be assumed structurally – 2∅14A400C. The supporting working reinforcement is to be located in the Fr1 frameworks. This reinforcement has recommended quarter bay length and is abutted to the upper structural reinforcement.

The gird's shelves are reinforced with bent Fr2 frameworks.

When designing the gird of a civil or industrial building, it would be rational to position the longitudinal working reinforcement in accordance with the bending moments epure.

Thus, in the example of the industrial building gird's reinforcement, it is possible to place only 2∅22A400C with a cross-section area of 7,6 cm^2 along the whole length of the gird, while 2∅25A400C with a cross section area of 9,82 cm^2 it is possible to place only in the area of greatest bending moment.

To determine the boundaries of this reinforcement drop off, we construct the epures of the design bending moments and moments of the actual carrying capacity under the gird reinforcement diagram (fig. 3.6).

Then we determine the actual carrying capacity of the gird's different cross sections.

1. The cross section carrying capacity with entire bay reinforcement (A_s=17,42 $см^2$).

$$\xi = f_{yd}A_s \,/\, 0{,}8f_{cd}bd = 36{,}5\cdot 17{,}42 \,/\, 0{,}8\cdot 1{,}45\cdot 25\cdot 59 = 0{,}372; \quad \zeta = 0{,}851;$$

$$M_{sect} = \zeta f_{yd}A_s d = 0{,}851\cdot 36{,}5\cdot 17{,}42\cdot 59 = 31924 \text{ kNcm} = 319{,}24 \text{ kNm}.$$

2. The carrying capacity of the cross section with the reinforcement left 2∅22A400C (A_s=7,6 cm^2).

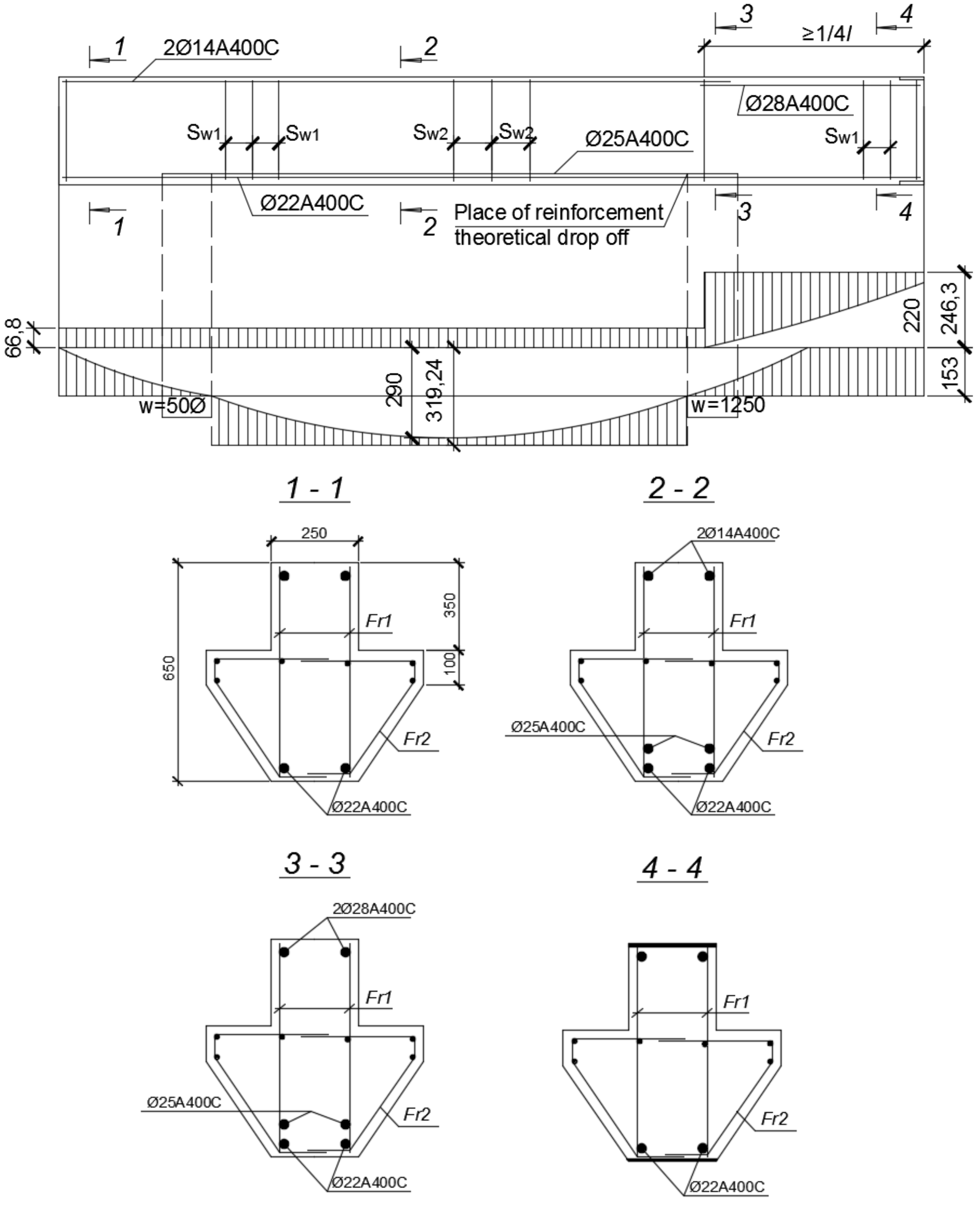


Figure 3.6 – Beam reinforcement

ξ = 36,5·7,6 / 0,8·1,45·25·59 = 0,162; ζ = 0,935;

M_{sect} = 0,935·36,5·7,6·59 = 15302 kNcm = 153 kNm.

3. The carrying capacity of the support cross section with 2∅28A400C (A_s=12,32 cm^2)

ξ = 36,5·12,32 / 0,8·1,45·25·61 = 0,254; ζ = 0,898;

M_{sect} = 0,898·36,5·12,32·61 = 24630 kNcm = 246,3 kNm.

4. The carrying capacity on the negative bending moment in the bay with 2∅14A400C (A_s=3,08 cm^2):

ξ = 36,5·3,08 / 0,8·1,45·25·61 = 0,064; ζ = 0,974;

M_{sect} = 0,974·36,5·3,08·61 = 6679 кНсм = 66,8 kNm.

The length of the reinforcement that is dropped off in the bay (2∅25A400C), exceeds the theoretical length by value w = 50∅ = 50·25 = 1250 mm from each edge.

4 CALCULATION OF A BUILDING'S COLUMN

The algorithm for calculating the columns of civil and industrial buildings is the same. As an example, we consider the calculation of an industrial building column.

4.1 The column Loading

To calculate, we accept the following additional data:

– the number of floors n = 4;

– each floor height $H_{пов}$ = 3,6 m;

– location of construction – the city of Kharkiv

– covering type – roofless;

– the building service life – 50 years.

According to DBN V.1.2-2:2006 Loads and effects, the operating snow load magnitude for the city of Kharkiv s_0 = 1,6 kN/м^2.

For the building's service life of 50 years, the reliability index for the snow load γ_f = 1,04.

The loading area from which the load on the column is generated,

$A = L$ x l = 6,4·6 = 38,4 m^2.

The column design load is determined at the floor level of the first floor.

The column load should be calculated in tabular form (Table 4.1).

Table 4.1 – Collecting loads on the column

№ in ord.	Load type	Characteristic load value, kN/m^2	Relia bility index γ_f	Design load value, kN/m^2
	A) Constant			
1	Rolled covering 0,1·38,4	3,84	1,3	5,0
2	Cement slurry (δ=2 см, ρ=20 kN/m^3) 0,02·20·38,4	15,36	1,3	20,0
3	Heat insulation and vapor barrier (δ=10 cm, ρ=6 kN/m^3) 0,1·6·38,4	23,04	1,3	30,0
4	Covering slabs 2,4·38,4	92,16	1,1	101,4
5	The weight of three floor slabs structures (ref. to table 3.1) 3,784·3·38,4	-	-	436,0
6	The weight of the girds' lower parts on all floors (0,55·0,35–0,15·0,2)25·6,4·4	104,0	1,1	114,4
7	The column own weight 0,4·0,4·3,6·25·4	57,6	1,1	63,4
	Total constant			770,2
	B) Variable			
8	– on floor slabs $v{\cdot}A{\cdot}(n{-}1)$ 9,6·38,4·3	-	-	1106
9	– on the covering (snow) 1,6·38,4	61,4	1,0	61,4
	Total variable			1 167,4
	Total			1 938

4.2 The column design calculation

The column design length for a multistorey building is assumed as $l_0 = H_{fl.}$= 3,6 m. The column should be prefabricated from C16/20 grade concrete.

Rebar – A400C grade.

The value of random eccentricity is assumed as the greater of the values

$e_i = l_0/600 = 360/600 = 0,6$ cm;

$e_i = h/30 = 40/30 = 1{,}33$ cm;

$e_i = 1$ cm.

Assume $e_i = 1{,}33$ cm.

Radius of section inertia $i = 0{,}289h = 0{,}289 \cdot 40 = 11{,}56$ cm.

The column flexibility $\lambda = l_0/i = 360/11{,}56 = 31{,}14$.

Relative axial force $n = N/A_c f_{cd} = 1938/1600 \cdot 1{,}15 = 1{,}053$.

Interfacial flexibility

$$\lambda_{lim} = \frac{20ABC}{\sqrt{n}} = \frac{20 \cdot 0{,}7 \cdot 1{,}1 \cdot 0{,}7}{\sqrt{1{,}053}} = 10{,}5 < \lambda = 31{,}14,$$

deformations of another order should be taken into account.

The cross section rigidity (for previous rebar amount $0{,}01A_c$)

$$EI = K_c E_{cd} I_c + 0{,}01 E_s A_c (0{,}5h - a)^2 ,$$

where: $K_c = 0{,}3 / (1 + 0{,}5\varphi_{ef})$; for the given creep coefficient $\varphi_{ef} = 2{,}2$

$$K_c = 0{,}3 / (1+0{,}5 \cdot 2{,}2) = 0{,}143;$$

$EI = 0{,}143 \cdot 2000 \cdot 40^4 / 12 + 0{,}01 \cdot 21\ 000 \cdot 1\ 600(20 - 4)^2 = 1{,}47 \cdot 10^8$ kNcm^2.

Critical force

$$N_B = \pi^2 EI / l_0^2 = 3{,}14^2 \cdot 1{,}47 \cdot 10^8 / 360^2 = 11\ 183 \text{ kN}.$$

Residual value of design excentricity

$$e_0 = e_i \left(1 + \frac{\beta}{\frac{N_B}{N}} \right),$$

where $\beta = \pi^2/c_0$; $c_0 = 8$ for the absence of transverse loading;

$$\beta = 3{,}14^2 / 8 = 1{,}232;$$

$$e_0 = 1{,}33 \left(1 + \frac{1{,}232}{\frac{11183}{1938}} \right) = 1{,}67 \text{ cm}.$$

The nucleus point coordinate $r = h / 6 = 40 / 6 = 6{,}67$ cm.

$$e = e_0 + 0{,}5h - a = 1{,}67 + 20 - 4 = 17{,}67 \text{ cm}.$$

Due to $e_0 < r$, we perform calculation using the first form of equilibrium (the entire cross section is compressed, $A_s = 0$).

$$A_s' = \frac{Ne - f_{cd}bh(0.5h - a)}{f_{yd}(d - d')} = \frac{1938 \cdot 17{,}67 - 1{,}15 \cdot 40 \cdot 40(20 - 4)}{36{,}5(36 - 4)} = 4{,}11 \text{ cm}^2.$$

The column can be deformed in any direction, so we accept the symmetrical reinforcement of $A_s^I = A_s = 4{,}11$ cm^2.

For general reinforcement of the column section we accept 4Ø18A400C ($A_s = 10{,}18$ cm^2).

The cross-section is accepted under condition of welding Ø6A240C with a step of 300 mm < 20Ø = 360 mm.

4.3 Design of the first floor prefabricated column

The first floor floor column has a larger length than on the other floors.

Thus, the lower part of the column is embedded in the foundation pocket to a depth of $\geq 1.5\ h_к$, while the upper part of the column extends to the abutment with the column of the second floor. This joint is located at an altitude of 600...800 mm from the level of the top of the floor slab.

The abutment area is reinforced by indirect reinforcement grid (in this course project 4 grids are accepted).

The column's consoles work in bend due the girds pressure.

The console rebar is accepted structurally (fig. 4.1).

The column of a civil building is calculated based on the same algorithm as an industrial building.

Design features apply only to the column console.

The pattern of the reinforcement of the column console of a civil building is shown in figure 4.2.

5 DESIGNING A MONOLITHIC FOUNDATION FOR A PREFABRICATED COLUMN

For the design of the foundation, the data of the preliminary calculation of the column is used and additionally – the design resistance of the base ground (according to the task).

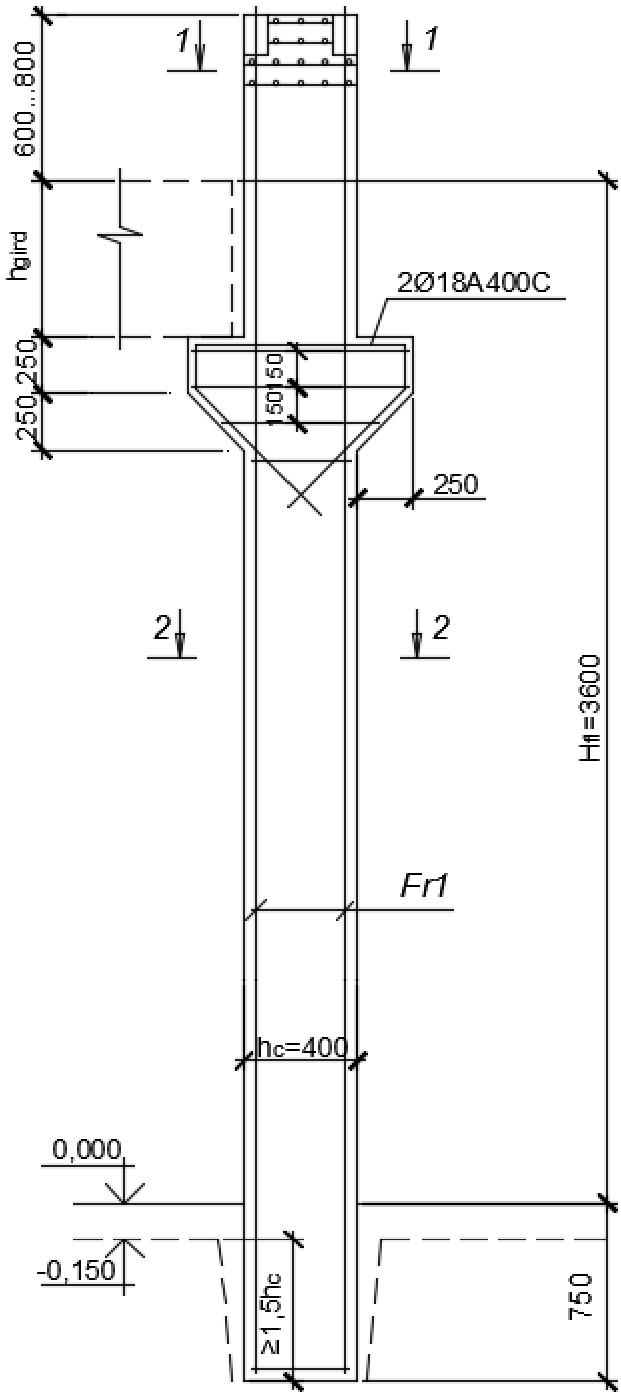


Figure 4.1 – Column reinforcement

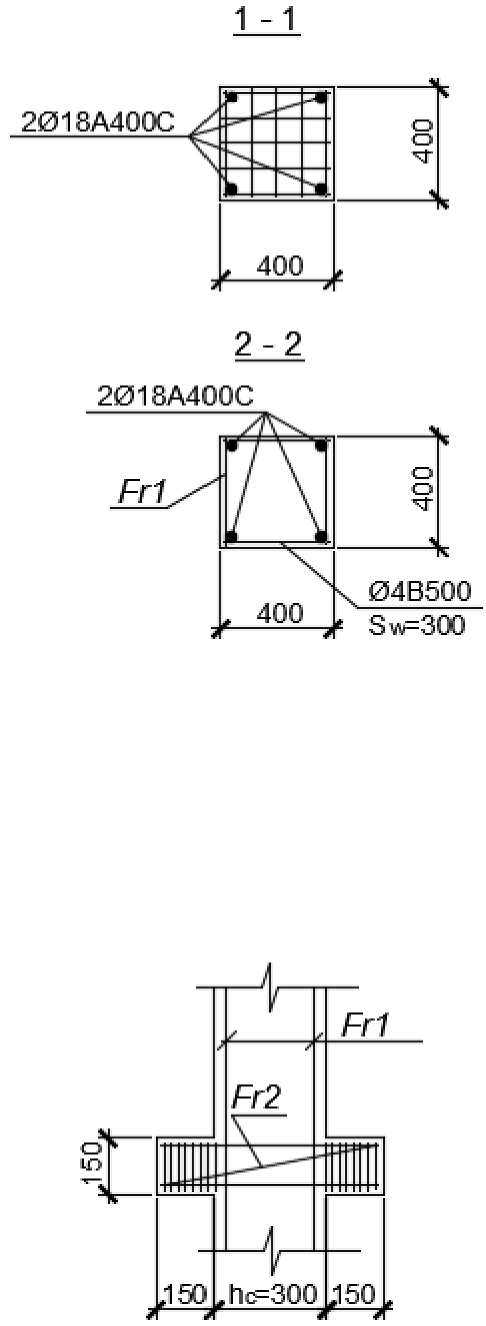


Figure 4.2 – The pattern of the reinforcement of the column console of a civil building

When designing the foundation, the sole area should be determined, the number and configuration of the steps and foundation reinforcement.

Overall dimensions of the foundation (base, height) are assumed as multiple of 300 mm.

The upper facet of the foundation is buried by 150 mm below the floor zero surface of the first floor.

5.1 Determination of the foundation base area

Now, we consider an example of calculating the foundation for a an industrial building column.

The foundation minimum height based on the accepted recommendations should be determined in advance.

Burying the column in foundation pocket $h_{bur} = 1{,}5h_{col} = 1{,}5{\cdot}40 = 60$ cm.

50 mm is reserved for the grout application From the column butt end to the bottom of the foundation pocket.

The thickness of the bottom (to the reinforcement) is assumed at least 200 mm. The distance from the reinforcement to the base of the foundation is assumed not less than 50 mm.

Thus, the minimum height of the foundation for an industrial building column, is $h = 600 + 50 + 200 + 50 = 900$ mm = 0,9 m.

Burying depth of the foundation base is

$$H = h + 150 \text{ mm} = 900 + 150 = 1050 \text{ mm} = 1{,}05 \text{ m}.$$

Design value of the foundation base area

$$A_f = N / \gamma_{fm} (f_0 - \rho_m H) ,$$

where: N – load on the foundation from the column (N = 1938 kN);

γ_{fm} –average reliability index with respect to loading of all the building elements (assumed as $\gamma_{fm} = 1{,}15$);

f_0 –the soil design resistance (assumed as $f_0 = 240$ kPa);

ρ_m – average specific weight of the concrete foundation and soil above it (assumed as $\rho_m = 20$ kN/м^3).

$$A_f = 1938 / 1{,}15(240 - 20{\cdot}1{,}05) = 7{,}69 \text{ m}^2.$$

The size of the base side $a = \sqrt{A_f} = \sqrt{7{,}69} = 2{,}77$ m.

Finally accepted dimensions of the base area

$$A_f = a^2 = 3{\cdot}3 = 9 \text{ m}^2.$$

Actual design pressure on the soil makes

$$p = N/A_{ф} + \rho_m H = 1938/9 + 20{\cdot}1{,}05 = 236 \text{ kPa}.$$

5.2 The Foundation Design Calculation

For the foundation design we accept concrete of C12/15 grade C12/15 (f_{ctd} = 0,73 MPa = 730 kPa), rebar of A400C grade (f_{yd} = 365 MPa).

When designing a foundation, the number and configuration of steps should be selected with reference to such recommendations.

1. The thickness of the foundation pocket walls is assumed at least 200 mm.

Taking into account the distance from the pocket upper surface to the column facet (75 mm), the first step will have the least outreach 275 mm.

2. The foundation lower step is performed as having height at least 300 mm.

3. The foundation is designed as two or three stepped.

We assume the foundation to be two-stepped (fig. 5.1).

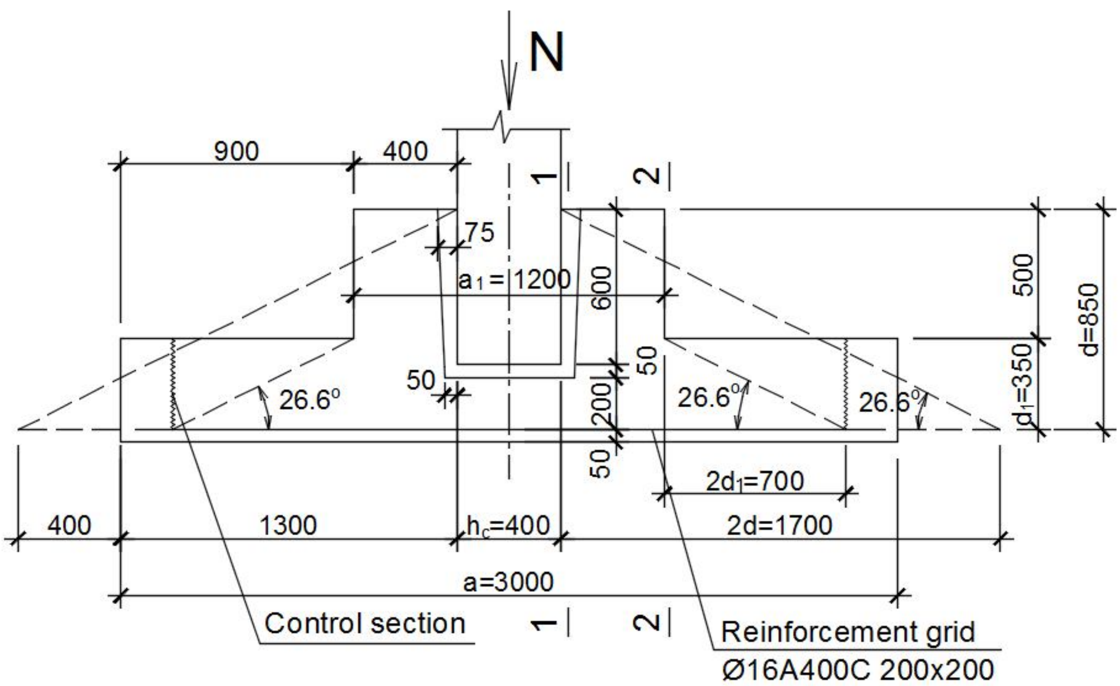


Figure 5.1 – Foundation construction (option 1)

The rebar is determined based on calculation for bending in two design cross sections: $M_{1\text{-}1} = 0{,}125pa(a - h_к)^2 = 0{,}125{\cdot}236{\cdot}3(3 - 0{,}4)^2 = 598{,}3$ kNm;

$$M_{2\text{-}2} = 0{,}125pa(a - a_1)^2 = 0{,}125{\cdot}236{\cdot}3(3 - 1{,}2)^2 = 286{,}4 \text{ kNm}.$$

The necessary amount of rebar

$$A_{s\,1} = M_{1\text{-}1}/0{,}9 f_{yd}d = 59\ 830/0{,}9{\cdot}36{,}5{\cdot}85 = 21{,}43\ \text{cm}^2;$$

$$A_{s\,2} = M_{2\text{-}2}/0{,}9 f_{yd}d_1 = 28\ 640/0{,}9{\cdot}36{,}5{\cdot}35 = 24{,}9\ \text{см}^2.$$

We assume the reinforcement as per the highest value: 15Ø16A400C (A_s = 30,16 см^2) with mesh size 200 mm × 200 mm.

5.3 Inspection of the foundation strength by impact-puncture test

The impact-puncture strength is checked only on a control section located at a distance $2d_1$ = 700 mm from the edge of the upper step (the control section at a distance of 2d from the column facet is beyond the foundation base).

The perimeter of the control section u = 2,6·4 = 10,4 m = 1,040 cm.

Weight of the upper step of the foundation G_1 = 1,2·1,2·0,5·20 = 14,4 kN.

The total pressure force on the foundation lower step

$$V_{Ed} = N + G_1 = 1938 + 14{,}4 = 1952{,}4\ \text{kN}.$$

The base pressure force within the boundaries of the control perimeter, is directed upwards

$$\Delta V_{Ed} = p{\cdot}b^2 = 236{\cdot}2{,}6^2 = 1595{,}4\ \text{kN}.$$

Impact-puncture effort

$$V_{Ed\,red} = V_{Ed} - \Delta V_{Ed} = 1\ 952{,}4 - 1595{,}4 = 357\ \text{kN}.$$

The stress at the control cross section

$$v_{Ed\,\sigma} = V_{Ed\,red}\ /\ ud_1 = 357\ /\ 1\ 040{\cdot}35 = 0{,}0099\ \text{kN/см}^2;$$

the stress of the cross section resistance to impact-puncture

$$V_{Rd,c\,\sigma} = C_{Rd,c}K\sqrt[3]{100\,\rho_1 f_{ck}}\ \frac{2d}{c},$$

where $C_{Rd,c} = 0{,}1385$; $K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{350}} = 1{,}756 < 2.$

$$\rho_1 = A_s/ad = 30{,}16\ /\ 300{\cdot}35 = 0{,}00287;$$

$$\upsilon_{Rd.c\,\sigma} = 0{,}1385{\cdot}1{,}756\sqrt[3]{100\cdot 0{,}00287\cdot 11}\ \frac{2\cdot 35}{70} =$$

$$= 0{,}435\ \text{MPa} > v_{Ed\,\sigma} = 0{,}0099\ \text{MPa}.$$

Strength strength is guaranteed.

If the cross-sectional impact-puncture strength is insufficient, a larger height of the lower step is accepted or a foundation with three steps is designed in such a way that the control section is beyond the foot the foundation base area (see option in fig. 5.2).

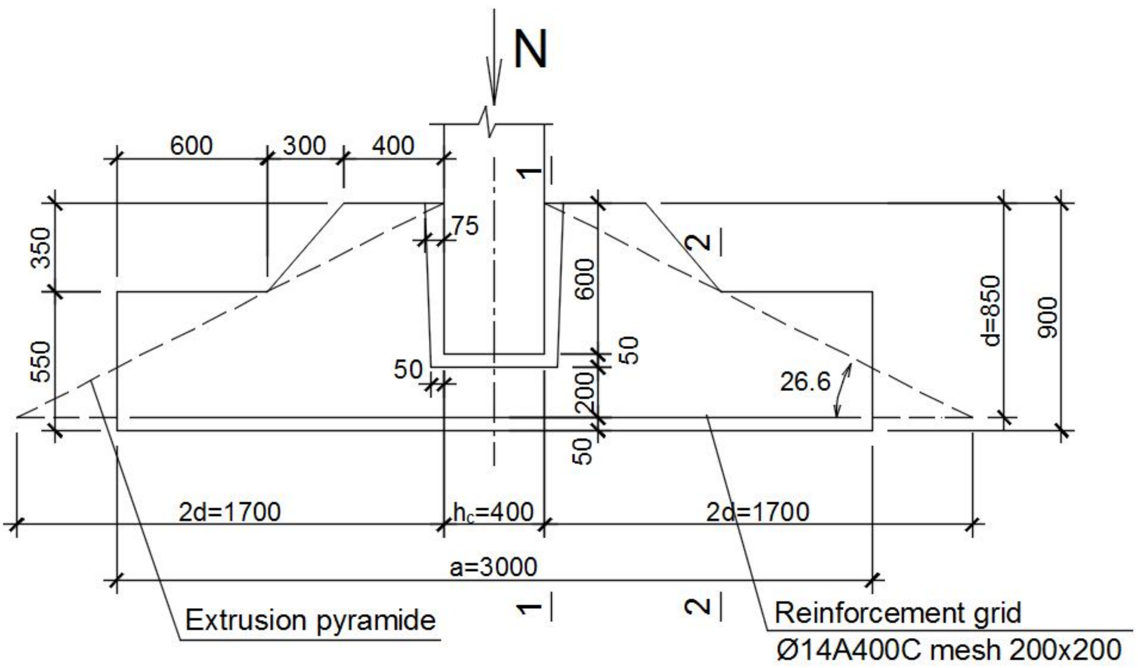


Figure 5.2 – Foundation construction (option 2)

6 GRAPHIC PART OF THE PROJECT

The graphic part for the prefabricated version of the building is performed on a sheet of A1 format with a monolithic version.

The graphical part shows:

– the layout of floor slab arrangement;

– cross-section of a building with prefabricated elements marking;

– formwork drawings, reinforcement patterns and reinforcement drawings of the gird, column and foundation for the column;

– specification of the gird, column and foundation rebar;

– an expense sheet for the specified elements rebar.

Below are examples of design drawings of calculated elements, specifications and rebar expense sheet (fig. 6.1 …6.3).

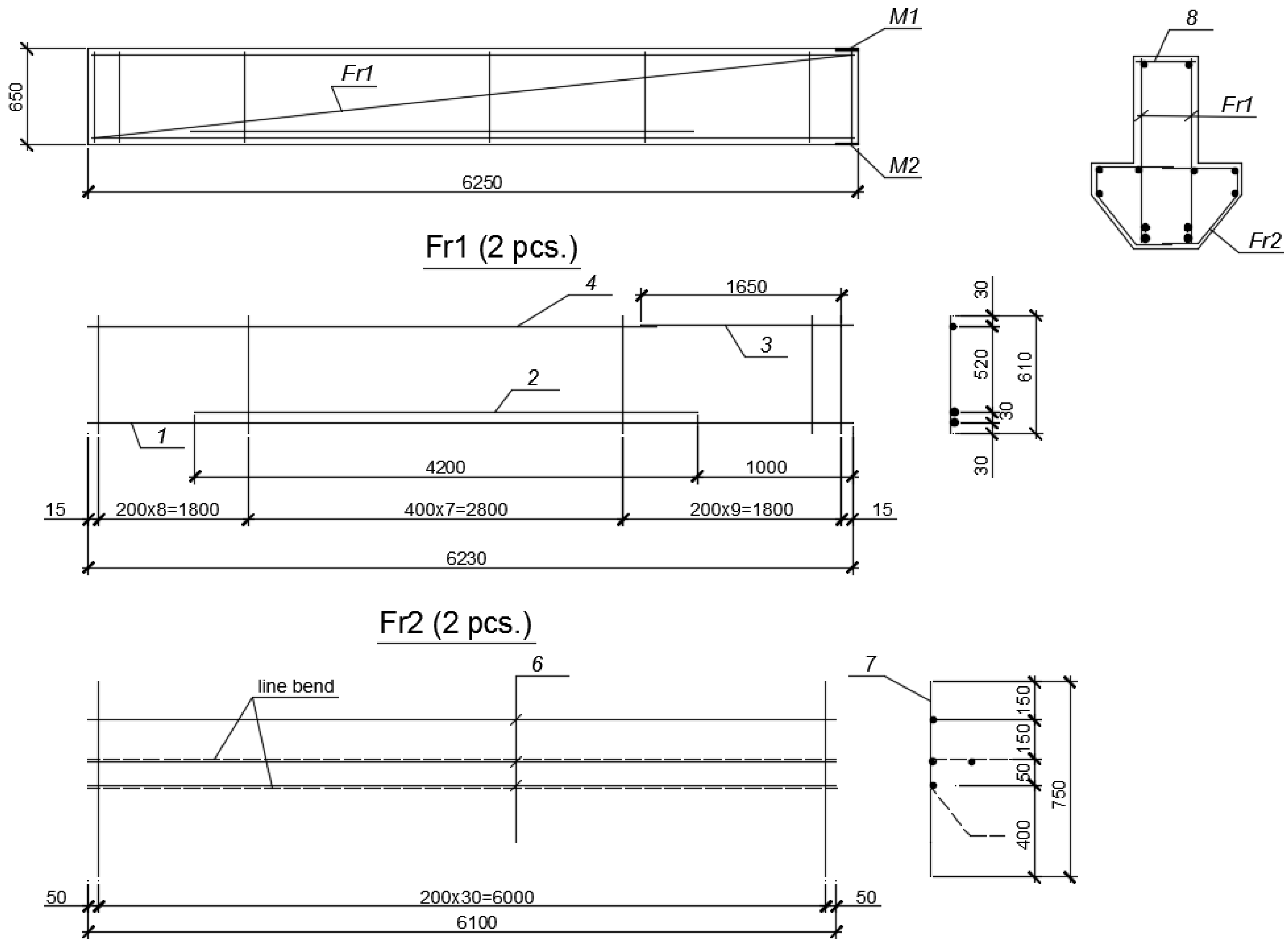


Figure 6.1 – Gird P1 Reinforcement scheme

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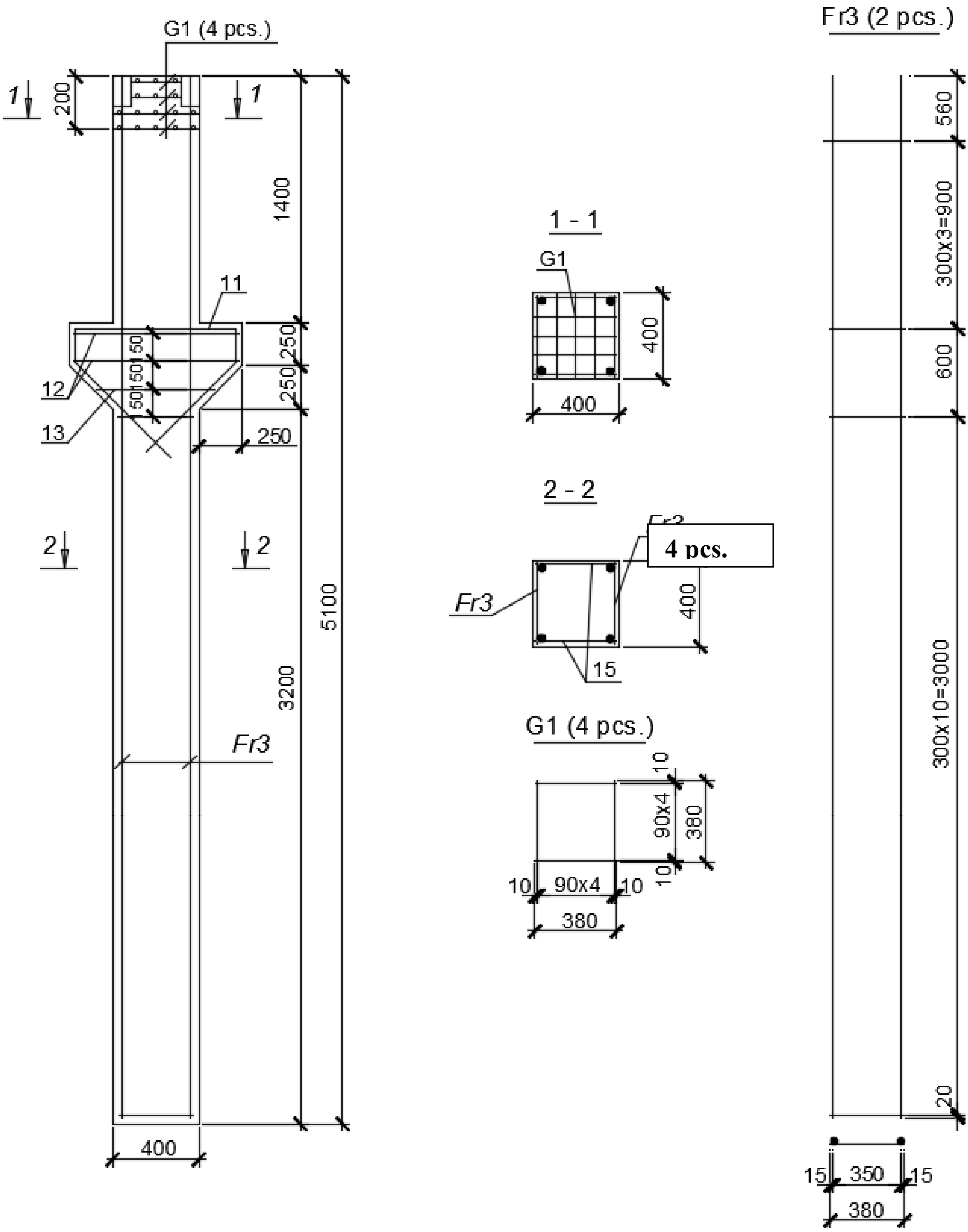

Figure 6.2 – Column C1 Reinforcement scheme

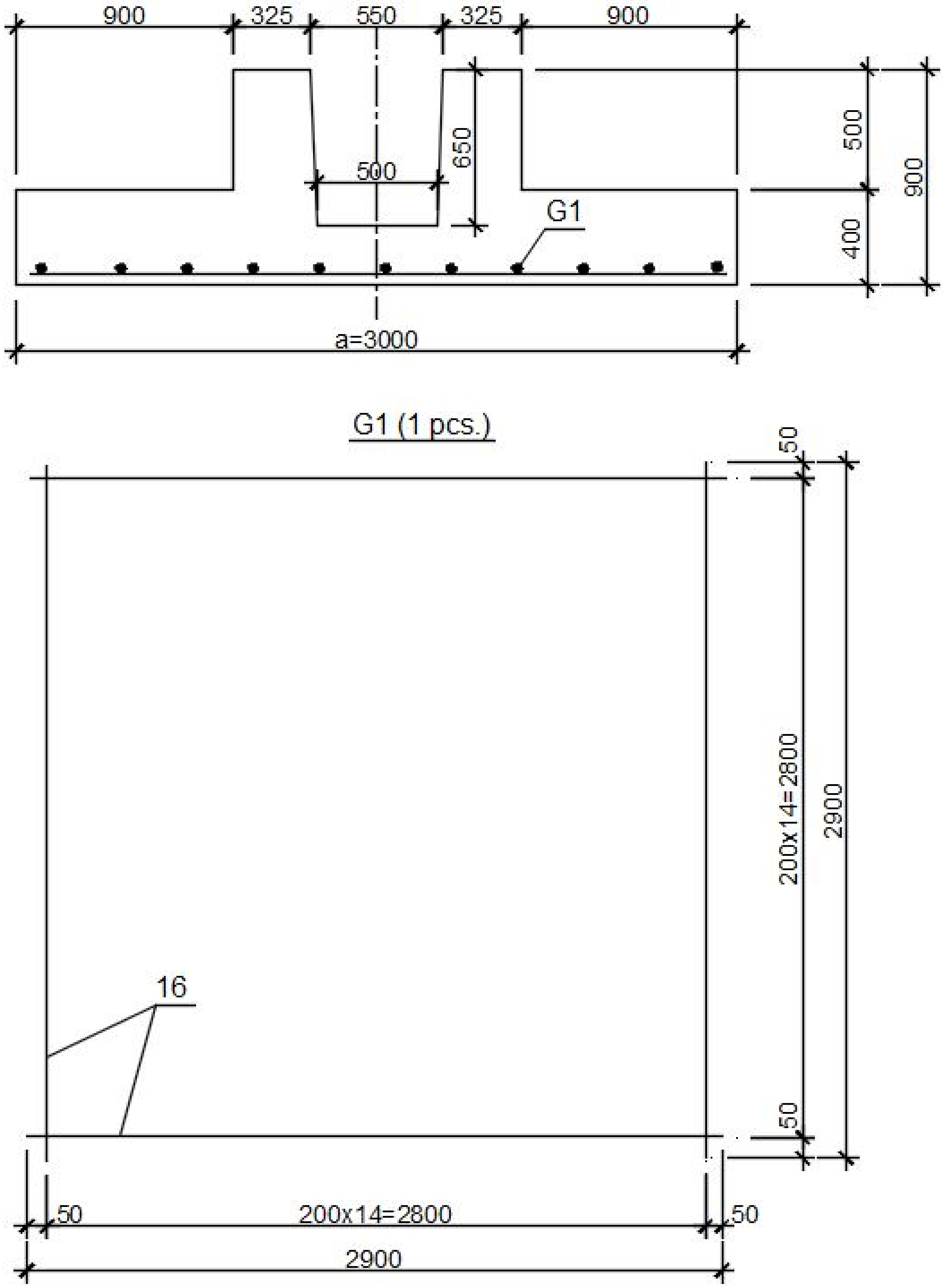

Figure 6.3 – Foundation FM1

Design. grade	Designation	Name	Q-ty	Unit weight, kg	Note
	PK-1.5	*Floor slab*	*8*		
	G1	*Reinforcement grid*	*1*	*42.7*	*42.7*
		Assemblies			
1	*DSTU 3760:2006*	*Ø12A4OOC, l=5730*	*8*	*5.1*	*40.8*
2	*DSTU ENV 10080*	*Ø3 B 500, l= 1460*	*23*	*0.08*	*1.84*
	G2	*Reinforcement grid*	*1*	*5.38*	*5.38*
		Assemblies			
3	*DSTU ENV 10080*	*Ø B500, l=1440*	*23*	*0.079*	*1.82*
4	*DSTU ENV 10080*	*Ø B500, l=5730*	*8*	*0.32*	*3.56*
	Fr 1	*Flat framework*	*8*	*0.27*	*2. 16*
		Assemblies			
5	*DSTU ENV 10080*	*Ø B500, l=200*	*16*	*0.011*	*0.1*
6	*DSTU ENV 10080*	*Ø B500, l= 1550*	*2*	*0.085*	*0.17*
		Additional data			
15	*60*	*65*	*10*	*15*	20

Figure 6.4 – Specification of the reinforcement items

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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 α_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 Ø8…22 Ø25…40	 435 415	 300 300	 435 415	21
B500	415	300	375	19

APPENDIX D

Value of coefficients α_m, ξ *ma* ζ

ξ	ζ	α_m	ξ	ζ	α_m	ξ	ζ	α_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	–	–	–

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

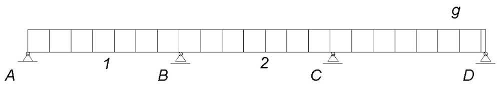
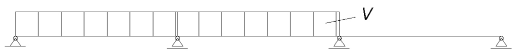
APPENDIX E

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 F

Bending moments $M = (\alpha g + \beta v)l_0^2$ and transverse forces $V = (\gamma g + \delta v)l_0$ of a multi-bay beam

№ in ord.	Loading scheme		Bay moments		Moments at support			Transverse forces		
			M_1	M_2	M_B	M_C		V_A	$V_B^{\text{лів.}}$	$V_B^{\text{пр.}}$
1		α	0,08	0,025	–0,1	–0,1	γ	0,4	–0,6	0,5
2		β	0,101	–0,05	–0,05	–0,05	δ	0,45	–0,55	0
3			–	–	–0,117	–0,033		0,383	–0,617	0,58

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з навчальної дисципліни

«ЗАЛІЗОБЕТОННІ КОНСТРУКЦІЇ»

Розділ 2
Проектування збірних елементів перекриття,
колон і фундаментів будівлі з неповним каркасом

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(англ. мовою)

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

Відповідальний за випуск *В. С. Шмуклер*
За авторською редакцією
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вул. Маршала Бажанова, 17, Харків, 61002.
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