

HANOI UNIVERSITY OF CIVIL ENGINEERING
DEPARTMENT OF CONCRETE STRUCTURES



REINFORCED CONCRETE STRUCTURES I

MINI-PROJECT

**DESIGN OF CAST-IN-SITU REINFORCED CONCRETE
ONE-WAY SLABS**

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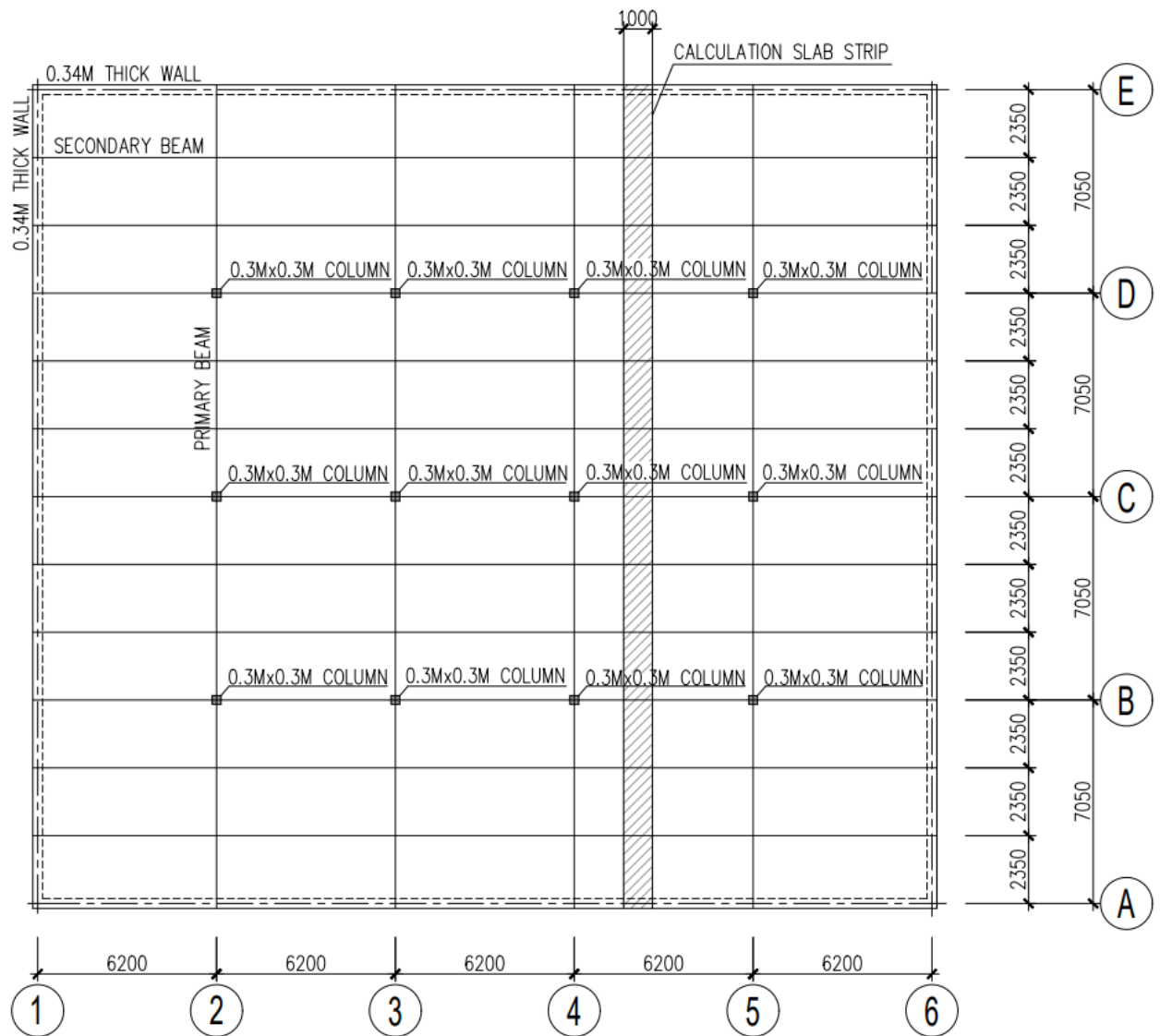
I. DESIGN DATA

1. Slab layout is shown in *Figure 1.1*.
2. The center-to-center distances between the beams along x-direction and to the boundary walls are $l_1 = 2,35m$. There are columns with cross section of $b_c \times h_c = 300mm \times 300mm$. The center-to-center spans between the columns in x-direction and between the second column and the boundary wall are $l_2 = 6,20m$. The boundary walls with thickness $b_t = 340mm$.
3. Slab is in civil building, with finishing layers as shown in *Figure 1.1*. The standard (unfactored) imposed live load $P^{tc} = 7,45 kN/m^2$ with the safety factor $n = 1,2$.
4. Materials: Concrete is of compressive strength class of B15, slab reinforcement and transverse reinforcement of beams is in class C-I, longitudinal reinforcement of beams is in class C-II. The design strength as follow:

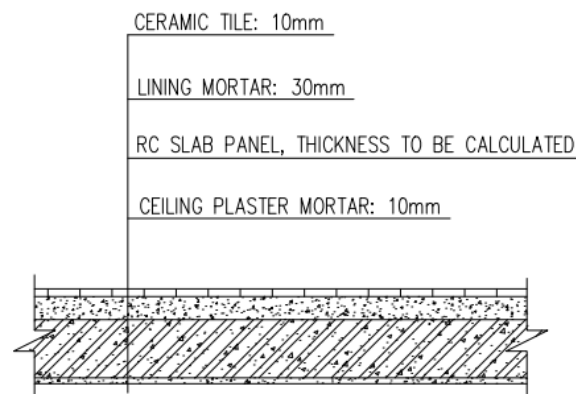
B15 concrete: $R_b = 8,5MPa$, $R_{bt} = 0,75MPa$, $E_b = 23 \times 10^3 MPa$.

C-I reinforcement: $R_s = 225MPa$, $R_{sc} = 225MPa$, $R_{sw} = 175MPa$, $E_s = 21 \times 10^4 MPa$.

C-II reinforcement: $R_s = 280MPa$, $R_{sc} = 280MPa$, $R_{sw} = 225MPa$, $E_s = 21 \times 10^4 MPa$.



SLAB LAYOUT



SLAB LAYERS

Figure 1.1. Slab layout and Slab layers

II. CALCULATION OF PLATE

2.1. Analysis of structural system

Ribbed slab supported by beams along 2 directions. The beams located on axis 2, 3, 4, 5 are primary beams, perpendicular to which are secondary beams. The sides of each slab panel are: $l_1 = 2350mm$, $l_2 = 6200mm$. The aspect ratio of slab panel is $\frac{l_2}{l_1} = \frac{6200}{2350} = 2,64 > 2$.

Hence, this is one-way slab.

2.2. Pre-select components dimensions

Slab thickness:

$$h_b = \frac{D}{m} \times l_1 = \frac{1,1}{35} \times 2350 = 73,86mm . \text{ Assume } h_b = 80mm.$$

whereas $D = 1,1$ with moderate loads, $m = 35$ with continuous slab panel.

Cross-sectional sizes of secondary beam:

$$h_{dp} = \frac{1}{m_{dp}} \times l_2 = \frac{1}{14} \times 6200 = 442,86mm . \text{ Assume } h_{dp} = 450mm, b_{dp} = 220mm.$$

Cross-sectional sizes of primary beam:

*Primary beam span equals to distance between 2 columns, 7050mm.

$$h_{dc} = \frac{1}{m_{dc}} \times l = \frac{1}{11} \times 7050 = 640,91mm . \text{ Assume } h_{dc} = 700mm, b_{dc} = 300mm.$$

Strengthening plate/Wall bracings with cross-sectional area of $b_g \times h_g = 340 \times 340mm$.

2.3. Calculation diagram

A panel strip with the width of $b_l = 1m$ perpendicular to the secondary beams is assumed to be separated from the slab and will be calculated as a continuous beam-type slab.

The calculation spans of slab panel strip are as follows:

Outer spans:

$$l_{ob} = l_{1b} - \frac{b_{dp}}{2} - \frac{b_t}{2} + \frac{h_b}{2} = 2,35 - \frac{0,22}{2} - \frac{0,34}{2} + \frac{0,08}{2} = 2,11m$$

Interior spans:

$$l_o = l_1 - b_{dp} = 2,35 - 0,22 = 2,13m$$

Span differences:

$$\frac{2,13 - 2,11}{2,13} \times 100\% = 0,94\%$$

2.4. Design loads

Dead load is calculated and presented in Table 2.1.

Table 2.1. Calculation of dead load

<i>Layers of plate</i>	<i>Standard (unfactored) value (kN/m²)</i>	<i>Safety factor</i>	<i>Design value (kN/m²)</i>
- Lining brick of 10mm, $\gamma = 20\text{kN/m}^3$	$0,01 \times 20 = 0,200$	1,1	0,220
- Lining mortar of 30mm, $\gamma = 18\text{kN/m}^3$	$0,03 \times 18 = 0,540$	1,3	0,702
- Reinforced concrete slab of 80mm, $\gamma = 25\text{kN/m}^3$	$0,08 \times 25 = 2,000$	1,1	2,200
- Plastering mortar, $\gamma = 18\text{kN/m}^3$	$0,01 \times 18 = 0,180$	1,3	0,234
Total	2,920		3,356

The total round off value $g_b = 3,36\text{kN/m}^2$

Live load $p_b = P^c \cdot n = 7,45 \times 1,2 = 8,94\text{kN/m}^2$

Total load $q_b = g_b + p_b = 3,36 + 8,94 = 12,3\text{kN/m}^2$

For 1m wide slab panel strip $b_1 = 1\text{m}$ has $q_b = 12,3 \times 1 = 12,3\text{kN/m}$.

2.5. Internal forces

Following the plastic scheme model:

- Bending moments at the outer span and the second support:

$$M_{nh} = M_{g2} = \pm \left(\frac{q_b \cdot l_{ob}^2}{11} \right) = \pm \left(\frac{12,3 \times 2,11^2}{11} \right) = \pm 4,978\text{kNm}$$

- Bending moments at the interior spans and interior supports:

$$M_{nhg1} = M_{g1} = \pm \left(\frac{q_b l_o^2}{16} \right) = \pm \left(\frac{12,3 \times 2,13^2}{16} \right) = \pm 3,488\text{kNm}$$

- Shear forces: $Q_A = 0,4q_b l_{ob} = 0,4 \times 12,3 \times 2,11 = 10,381\text{kN}$

$$Q_B^t = 0,6q_b l_{ob} = 0,6 \times 12,3 \times 2,11 = 15,572\text{kN}$$

$$Q_B^p = 0,5q_b l_o = 0,5 \times 12,3 \times 2,13 = 13,1\text{kN}$$

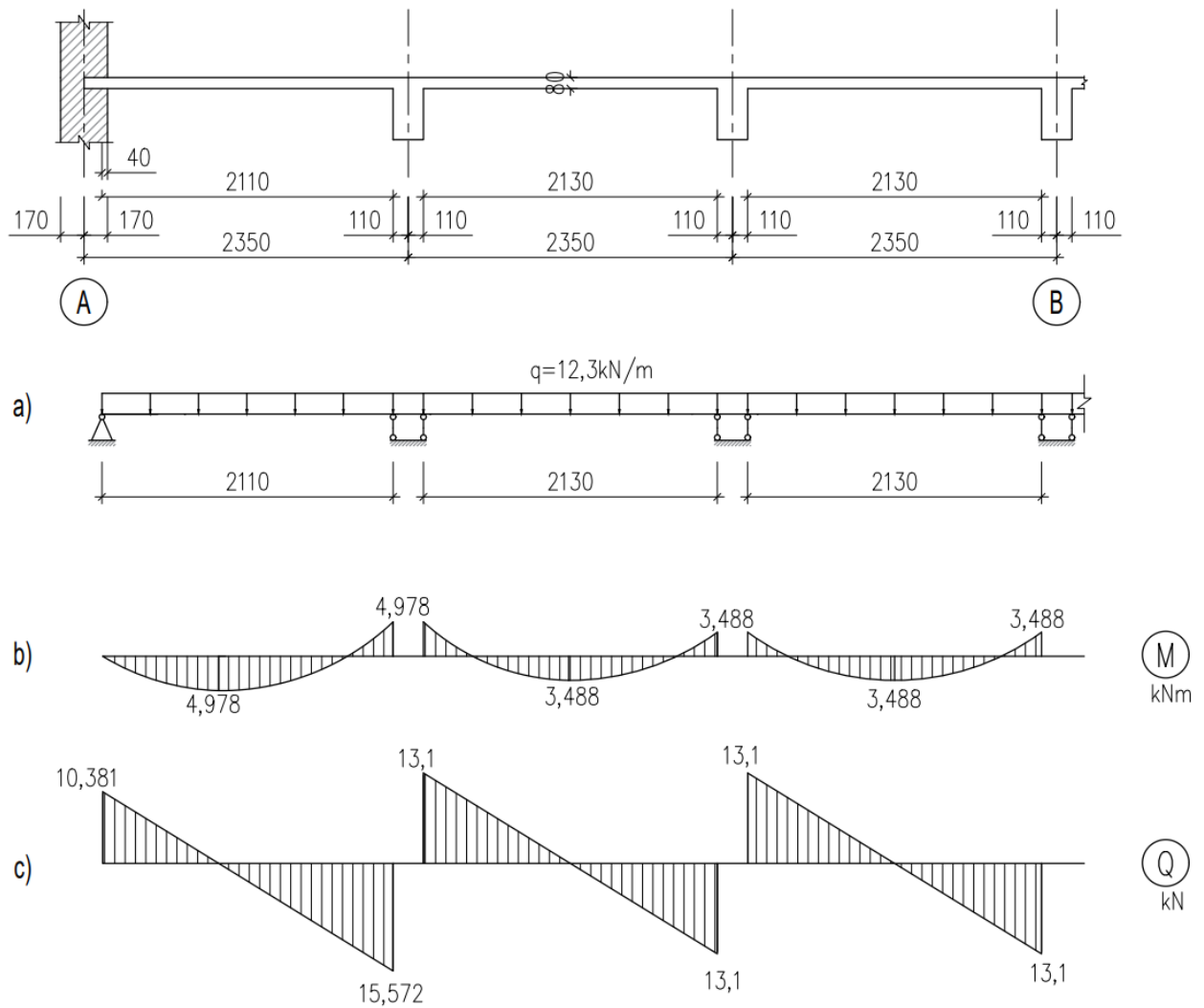


Figure 2.1. Calculation diagram and internal force diagram of slab strip

a) Calculation diagram; b) Bending moment diagram; c) Shear force diagram

2.6. Calculation of bending moment bearing reinforcement:

Concrete compressive strength class B15 with $R_b = 8,5 \text{ MPa}$, C-I class reinforcement with $R_s = 225 \text{ MPa}$.

Slab is analyzed by plastic theorem with the limit of relative effective depth of compression zone $\xi_{pl} = 0,3$ and the corresponding coefficient of $\alpha_{pl} = 0,225$.

Select $a = 15 \text{ mm}$ for all section, the effective depth of slab panels is:

$$h_0 = h_b - a = 80 - 15 = 65 \text{ mm}$$

At the second support and outer span, with $M = 4,978 \text{ kNm}$ (convert into $4,978 \times 10^6 \text{ Nmm}$):

$$\alpha_m = \frac{M}{R_b b h_0^2} = \frac{4,978 \times 10^6}{8,5 \times 1000 \times 65^2} = 0,139 < \alpha_{pl} = 0,255$$

$$\zeta = 0,5 \times (1 + \sqrt{1 - 2 \times \alpha_m}) = 0,5 \times (1 + \sqrt{1 - 2 \times 0,139}) = 0,925$$

$$A_s = \frac{M}{R_s \zeta h_0} = \frac{4,978 \times 10^6}{225 \times 0,925 \times 65} = 368 \text{ mm}^2$$

$$\mu\% = \frac{A_s}{b_1 h_0} = \frac{368}{1000 \times 65} \times 100 = 0,57\%$$

Select 8mm-diameter rebars, $a_s = 50,27 \text{ mm}^2$, the distance between rebars is:

$$s = \frac{b_1 a_s}{A_s} = \frac{1000 \times 50,27}{368} \approx 136,6 \text{ mm} \rightarrow \text{Select } \phi 8, s = 130 \text{ mm}.$$

At interior supports and mid-span of interior spans, $M = 3,488 \text{ kNm}$, $\alpha_m = 0,097$; $\zeta = 0,949$; $A_s = 252 \text{ mm}^2$.

Select diameter of reinforcement 6mm, $a_s = 28,3 \text{ mm}^2$, $s = 112 \text{ mm} \rightarrow \text{Select } \phi 6, s = 110 \text{ mm}.$

The actual effective depth is:

$$h_o = 80 - 10 - 0,5 \times 8 = 66 \text{ mm}.$$

$\rightarrow$ The assumed value for calculation $h_o = 65 \text{ mm}$ is conservative.

Since $p_b/g_b = 8,94/3,36 = 2,66 < 3$, the coefficient $\nu = 0,25$; the overhanging reinforcement for negative bending moments will stop at a distance from the secondary edge of $\nu l_o = 0,25 \times 2,13 = 0,53 \text{ m}$ and at a distance from the axis line of secondary beams of: $\nu l_o + 0,5 b_{dp} = 0,53 + 0,5 \times 0,22 = 0,64 \text{ m}.$

The overhanging rebars for negative bending moments can be cut and alternatively arranged with the remaining rebars. The overhanging of the shorter rebars from the beam edge is:

$$\frac{1}{6} \times l_o = \frac{1}{6} \times 2,13 = 0,36 \text{ m}$$

The distance between the end of the shorter rebars and the axis line of secondary beam is:

$$\frac{1}{6} \times l_o + 0,5 \times b_{dp} = \frac{1}{6} \times 2,13 + 0,5 \times 0,22 = 0,47 \text{ m}$$

The rebars for positive bending moment can also be cut off and alternatively arranged with the remaining rebars. The distance from the end of the cut off rebars to the edge of secondary beam is:

$$\frac{1}{8} \times l_o = \frac{1}{8} \times 2,13 = 0,27m$$

The shear reinforcement is not required in slabs, since the concrete shear strength is already sufficient, such as:

$$Q_{bmin} = 0,625R_{bt}b_1h_0 = 0,625 \times 0,75 \times 1000 \times 65 = 30468N = 30,47kN$$

$$Q_{max} = Q_B^t = 15,572kN < Q_{bmin}.$$

2.7. Distribution reinforcement

Rebars for negative bending moment that are perpendicular to primary beams: choose $\phi 6$, $s = 200$ with cross-sectional area of 141 mm^2 per 1m width, greater than 50% of the area of main rebars for negative bending moment at interior supports, which is $0,5 \times 252 = 126 \text{ mm}^2$. These bars stop at a distance of $\frac{1}{4} \times l_o = \frac{1}{4} \times 2,13 = 0,53m$ from the primary beam edge and of $0,53 + 0,5 \times b_{dc} = 0,68m$ from the primary beam axis line.

The rebars perpendicular to the main rebars are $\phi 6$, $s = 250$ with a cross-sectional area of 141 mm^2 per 1m width, greater than 30% of those of main rebars for positive bending moment at mid-span of spans, which are $0,3 \times 368 = 110,4 \text{ mm}^2$ for end spans and $0,3 \times 252 = 75,6 \text{ mm}^2$ for interior spans.

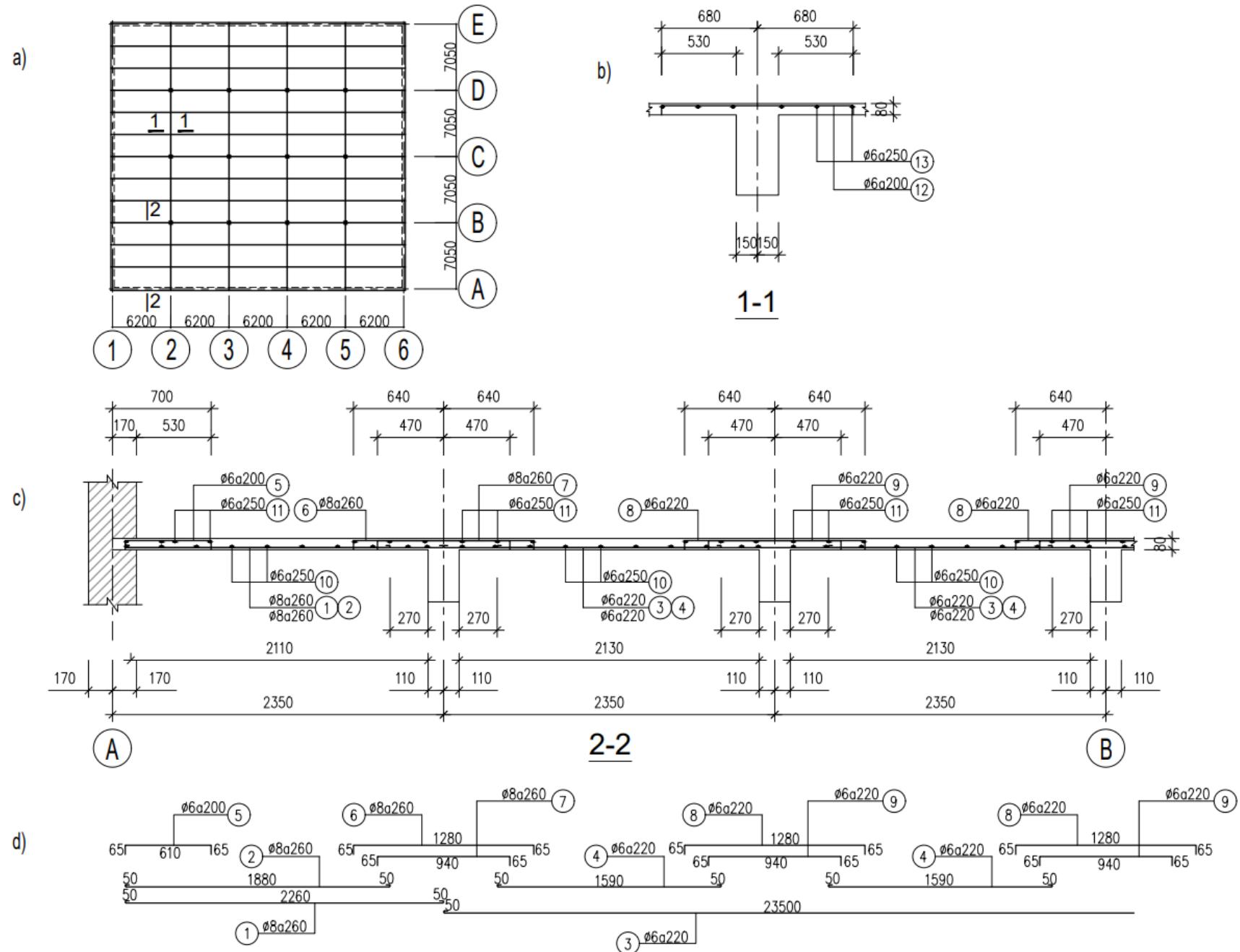


Figure 2.2. Reinforcement arrangement in slab panel

a) Layout; b) Section 1-1; c) Section 2-2; d) Rebar development.

a) Calculation diagram; b) Bending moment envelope; c) Shear force diagram

3.2. Design loads

Design dead loads include:

- Self-weight of beam (the 80mm thickness of the slab shall be subtracted)

$$g_{dp} = b_{dp} (h_{dp} - h_b) \gamma_n = 0,22 \times (0,45 - 0,08) \times 25 \times 1,1 = 2,239 \text{ kN/m}$$

- Dead loads transmitted from slab panels:

$$g_b \cdot l_1 = 3,36 \times 2,35 = 7,896 \text{ kN/m}$$

Total Dead loads:

$$g_p = g_{dp} + g_b l_1 = 2,239 + 7,896 = 10,135 \text{ kN/m}$$

Live loads transmitted from slab panels:

$$p_p = p_b l_1 = 8,94 \times 2,35 = 21 \text{ kN/m}$$

Total loads:

$$q_p = g_p + p_p = 10,135 + 21 = 31,135 \text{ kN/m}$$

Ratio:

$$\frac{p_p}{g_p} = \frac{21}{10,135} = 2,072$$

3.3. Internal forces

a. Bending moment

The ordinates of the bending moment envelope (positive moments)

$$\text{- End spans: } M^+ = \beta_1 q_p l_{pb}^2 = \beta_1 \times 31,135 \times 6,035^2 = \beta_1 \times 1133,975 \text{ kNm}$$

$$\text{- Interior spans: } M^+ = \beta_1 q_p l_p^2 = \beta_1 \times 31,135 \times 5,900^2 = \beta_1 \times 1083,809 \text{ kNm}$$

The ordinates of the bending moment envelope (negative moments)

$$M^- = \beta_2 q_p l_p^2 = \beta_2 \times 31,135 \times 5,9^2 = \beta_2 \times 1083,809 \text{ kNm}$$

Based on *Appendix A.11* in “*Design of cast-in-situ reinforced concrete slab and beam structures*” and the ratio $p_p/g_p = 2,072$, to get $k = 0,253$ and the coefficients β_1, β_2 . The calculated results are shown in *Table 3.1*

Table 3.1. Determination of bending moment envelope of secondary beams

Span, cross-section	β value		M value (kNm)	
	β_1	β_2	M^+	M^-
Outer span				
1st support	0		0	
1	0,065		73,708	
2	0,09		102,058	
0,425l	0,091		103,192	
3	0,075		85,048	
4	0,02		22,680	
2nd support – Cross-section 5		-0,0715		-77,492
Span no.2				
6	0,018	-0,0304	19,509	-32,948
7	0,058	-0,0094	62,861	-10,188
0,5l	0,0625		67,738	
8	0,058	-0,0064	62,861	-6,936
9	0,018	-0,0244	19,509	-26,445
Span no.3 – Cross-section 10		-0,0625		-67,738
Middle span				
11	0,018	-0,0233	19,509	-25,253
12	0,058	-0,0035	62,861	-3,793
0,5l	0,0625	-0,0035	67,738	-3,793

The cross-sections having zero bending moment on top (negative moment) are at a distance from the left of the second support of:

$$x = kl_{pb} = 0,253 \times 6,035 = 1,527m.$$

These sections having zero bending moment at bottom (positive moment) are at a distance from the support of:

$$\text{- End spans: } 0,15l_{pb} = 0,15 \times 6,035 = 0,905m$$

$$\text{- Interior span: } 0,15l_p = 0,15 \times 5,9 = 0,885m$$

b. Shear forces

$$Q_1 = 0,4q_pl_{pb} = 0,4 \times 31,135 \times 6,035 = 75,16kN$$

$$Q_2^t = 0,6q_pl_{pb} = 0,6 \times 31,135 \times 6,035 = 112,74kN$$

$$Q_2^p = Q_3 = 0,5q_pl_p = 0,5 \times 31,135 \times 5,9 = 91,848kN$$

The bending moment envelope and shear force diagram are shown in *Figure 3.1*.

3.4. Calculation of reinforcement for bending (longitudinal rebars)

Concrete compressive strength class B15 with $R_b = 8,5MPa$; CII class with $R_s = 280MPa$, $R_{sc} = 280MPa$ for longitudinal rebars.

Secondary beams are analysed by plastic theorem with the limit relative depth of compression zone $\xi_{pl} = 0,3$ and the corresponding coefficient $\alpha_{pl} = 0,255$.

a. For negative bending moment

The beam cross section is calculated as a rectangular with $b = 220mm$; $h = 450mm$

Assume $a = 30mm$, $h_0 = 450 - 30 = 420mm$.

At the second support, with $M = 77,492kNm$

$$\alpha_m = \frac{M}{R_b b h_0^2} = \frac{77,492 \times 10^6}{8,5 \times 220 \times 420^2} = 0,235 < \alpha_{pl} = 0,255$$

$$\zeta = \frac{1 + \sqrt{1 - 2\alpha_m}}{2} = \frac{1 + \sqrt{1 - 2 \times 0,235}}{2} = 0,864$$

$$A_s = \frac{M}{R_s \zeta h_0} = \frac{77,492 \times 10^6}{280 \times 0,864 \times 420} = 763mm^2$$

Check reinforcement ratio:

$$\mu\% = \frac{A_s}{b_{dp} h_0} = \frac{763}{220 \times 420} \times 100 = 0,83\%$$

At third support, with $M = 67,738kNm$:

$$\alpha_m = 0,205; \zeta = 0,884; A_s = 652mm^2, \mu\% = 0,71\%.$$

b. For positive moment

The beam cross section is calculated in T-shaped having flange in compression zone, the flange thickness $h_f = 80mm$.

Assume that $a = 30mm$, $h_0 = 450 - 30 = 420mm$.

The overhang length of flange does not exceed the minimum value of:

$$- (1/6)l_d = (1/6) \times 5,9 = 0,983m$$

- A half of the clear distance between the adjacent secondary beams:

$$0,5l_o = 0,5 \times 2,13 = 1,065m$$

(since $h_f > 0,1.h$, with $h = 450mm$ and the distance between transverse beams are

longer than those of longitudinal beams: $6,2m > 2,35m$)

Then $S_f \leq \min (0,983; 1,065)m = 0,983m$. Choose $S_f = 983mm$.

The effective width of flange: $b_f = b + 2S_f = 220 + 2 \times 983 = 2186mm$

Compute: $M_f = R_b b_f h_f (h_0 - 0,5 h_f)$

$$M_f = 8,5 \times 2186 \times 80 \times (420 - 0,5 \times 80) = 564,862 \times 10^6 Nmm.$$

$M_{max}^+ = 103,192kNm < M_f = 564,862kNm \rightarrow$ Neutral axis is within flange.

Calculation in rectangular $b = b_f = 2186mm$; $h = 450mm$; $a = 30mm$, $h_0 = 420mm$.

At end spans, with $M^+ = 103,192kNm$

$$\alpha_m = \frac{M}{R_b b_f h_0^2} = \frac{103,192 \times 10^6}{8,5 \times 2186 \times 420^2} = 0,031 < \alpha_{pl} = 0,255$$

$$\zeta = \frac{1 + \sqrt{1 - 2\alpha_m}}{2} = \frac{1 + \sqrt{1 - 2 \times 0,031}}{2} = 0,984$$

$$A_s = \frac{M}{R_s \zeta h_0} = \frac{103,192 \times 10^6}{280 \times 0,984 \times 420} = 892mm^2$$

Checking reinforcement ratio

$$\mu\% = \frac{A_s}{b_{dp} h_0} = \frac{892}{220 \times 420} \times 100 = 0,97\%$$

At interior spans, with $M = 67,738kNm$:

$\alpha_m = 0,021$; $\zeta = 0,989$; $A_s = 583mm^2$; $\mu = 0,63\%$.

3.5. Selection and arrangement of longitudinal reinforcement

Table 3.2. Longitudinal rebars arrangement of main sections of beam

Section	Outer span	Second support (B)	Span no.2	Third support (C)	Span no.3
Design A_s	892mm^2	763mm^2	583mm^2	652mm^2	583mm^2
Rebars	$2\Phi 20+1\Phi 20$	$2\Phi 18+1\Phi 18$	$2\Phi 16+1\Phi 16$	$2\Phi 16+1\Phi 18$	$2\Phi 16+1\Phi 16$
Rebars area	942mm^2	763mm^2	603mm^2	657mm^2	603mm^2

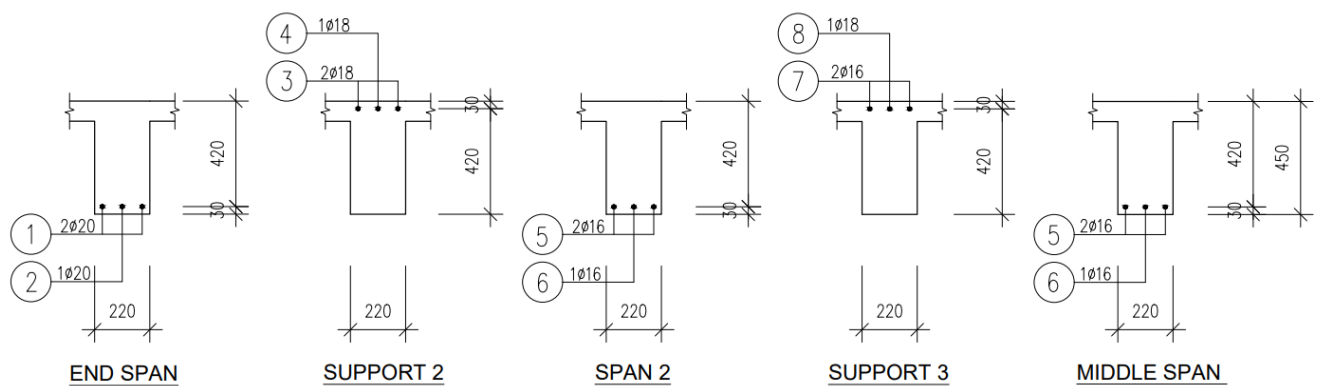


Figure 3.2. Longitudinal rebars arrangement of main sections of secondary beams

Recheck h_0 :

Select covering thickness $c = 20\text{mm}$. The actual effective depth of the beam is:

$$h_0 = h - c - \phi/2 = 450 - 20 - 20/2 = 420\text{mm}$$

The assumed value for calculation $h_0 = 420\text{mm}$ is conservative.

3.6. Calculation of reinforcement for shear

Concrete compressive strength class B15 with $R_b = 8,5\text{MPa}$; $R_{bt} = 0,75\text{MPa}$; C-I class reinforcement with $R_{sw} = 175\text{MPa}$ for transverse rebars.

Shear forces given from Figure 3.1 are:

$$Q_1 = 75,16\text{kN}; Q_2^T = 112,74\text{kN}; Q_2^P = 91,848\text{kN}.$$

Consider the segment at the right side of support 1, $Q_{max} = Q_1 = 75,16\text{kN}$ is used for calculation of transverse reinforcement.

The beam section is rectangular $b = 220\text{mm}$; $h = 450\text{mm}$; $h_0 = 420\text{mm}$.

Compute: $0,3R_b b h_0 = 0,3 \times 8,5 \times 220 \times 420 = 235620\text{N} = 235,62\text{kN}$.

$$Q_{max} \leq 0,3R_b b h_0$$

The beams are not failed by the principal compressive stress at the web!

Compute: $Q_{bmin} = 0,5R_{bt}bh_o = 0,5 \times 0,75 \times 220 \times 420 = 34650N = 34,65kN$.

Since $Q_{max} = 75,16 > Q_{bmin}$, transverse reinforcement is required.

Pre-select $\phi_w = 6mm$, $s = 200mm$, $n_w = 2$ (2 legs). The cross-sectional area of a layer of stirrups:

$$A_{sw} = \frac{n\pi\phi_w^2}{4} = \frac{2 \times 3,14 \times 6^2}{4} = 56,52mm^2$$

$$q_{sw} = \frac{R_{sw}A_{sw}}{s} = \frac{175 \times 56,52}{200} = 49,455 \text{ N/mm}$$

$$q_{swmin} = 0,25R_{bt}b = 0,25 \times 0,75 \times 220 = 41,25 \text{ N/mm}$$

$q_{sw} > q_{swmin} \rightarrow$ Detailing requirement of stirrups is satisfied.

Compute:

$$q_1 = q_p - 0,5p_p = 31,135 - 0,5 \times 21 = 20,635 \text{ kN/m}$$

$$\begin{aligned} Q_u(0) &= 2,5R_{bt}bh_o + 0,75q_{sw}h_o \\ &= 2,5 \times 0,75 \times 220 \times 420 + 0,75 \times 49,455 \times 420 \\ &= 188828N = 188,828kN \end{aligned}$$

$$\begin{aligned} Q_u(h_o) &= 1,5R_{bt}bh_o + 0,75q_{sw}h_o + q_1h_o \\ &= 1,5 \times 0,75 \times 220 \times 420 + 0,75 \times 49,455 \times 420 + 20,635 \times 420 \\ &= 128195N = 128,195kN \end{aligned}$$

$$\begin{aligned} Q_u(2h_o) &= 0,75R_{bt}bh_o + 1,5q_{sw}h_o + 2q_1h_o \\ &= 0,75 \times 0,75 \times 220 \times 420 + 1,5 \times 49,455 \times 420 + 2 \times 20,635 \times 420 \\ &= 100465N = 100,465kN \end{aligned}$$

$$\begin{aligned} Q_u(3h_o) &= 0,5R_{bt}bh_o + 1,5q_{sw}h_o + 3q_1h_o \\ &= 0,5 \times 0,75 \times 220 \times 420 + 1,5 \times 49,455 \times 420 + 3 \times 20,635 \times 420 \\ &= 91807N = 91,807kN \end{aligned}$$

$$Q_{u,1} = \min[Q_u(0); Q_u(h_o); Q_u(2h_o); Q_u(3h_o)] = 91,807kN > Q_{max} = 75,16kN$$

Hence, stirrups meet the shear bearing capacity requirements at critical sections.

Compute:

$$c_2 = \sqrt{\frac{1,5R_{bt} \times b \times h_o^2}{q_1}} = \sqrt{\frac{1,5 \times 0,75 \times 220 \times 420^2}{20,635}} = 1455mm$$

$$c_3 = \sqrt{\frac{1,5R_{bt} \times b \times h_o^2}{0,75q_{sw} + q_1}} = \sqrt{\frac{1,5 \times 0,75 \times 220 \times 420^2}{0,75 \times 49,455 + 20,635}} = 869,7mm$$

$c_2 = 1455mm > 2h_0 = 840mm \rightarrow$ Checking shear bearing capacity at section c_2 is not required.

$c_3 = 869,7mm > 2h_0 = 840mm. \rightarrow$ Checking shear bearing capacity at section c_3 is not required.

Choose $\phi_w = 6mm, s = 200mm, n_w = 2$.

Consider the segment at the left side of support 2, $Q_{max} = Q_2^T = 112,74kN$ is used for calculation of transverse reinforcement.

The beam section is rectangular $b = 220mm; h = 450mm; h_0 = 420mm$.

Compute: $0,3R_b b h_0 = 0,3 \times 8,5 \times 220 \times 420 = 235620N = 235,62kN$.

$$Q_{max} \leq 0,3R_b b h_0$$

The beams are not failed by the principal compressive stress at the web!

Compute: $Q_{bmin} = 0,5R_{bt} b h_0 = 0,5 \times 0,75 \times 220 \times 420 = 34650N = 34,65kN$.

Since $Q_{max} = 112,74 > Q_{bmin}$, transverse reinforcement is required.

Pre-select $\phi_w = 6mm, s = 100mm, n_w = 2$ (2 legs). The cross-sectional area of a layer of stirrups:

$$A_{sw} = \frac{n\pi\phi_w^2}{4} = \frac{2 \times 3,14 \times 6^2}{4} = 56,52mm^2$$

$$q_{sw} = \frac{R_{sw} A_{sw}}{s} = \frac{175 \times 56,52}{100} = 98,91 \text{ N/mm}$$

$$q_{swmin} = 0,25R_{bt} b = 0,25 \times 0,75 \times 220 = 41,25 \text{ N/mm}$$

$q_{sw} > q_{swmin} \rightarrow$ Detailing requirement of stirrups is satisfied.

Compute:

$$q_1 = q_p - 0,5p_p = 31,135 - 0,5 \times 21 = 20,635 \text{ kN/m}$$

$$\begin{aligned} Q_u(0) &= 2,5R_{bt} b h_0 + 0,75q_{sw} h_0 \\ &= 2,5 \times 0,75 \times 220 \times 420 + 0,75 \times 98,91 \times 420 \\ &= 204407N = 204,407kN \end{aligned}$$

$$\begin{aligned} Q_u(h_0) &= 1,5R_{bt} b h_0 + 0,75q_{sw} h_0 + q_1 h_0 \\ &= 1,5 \times 0,75 \times 220 \times 420 + 0,75 \times 98,91 \times 420 + 20,635 \times 420 \\ &= 143773N = 143,773kN \end{aligned}$$

$$\begin{aligned}
Q_u(2h_0) &= 0,75R_{bt}bh_0 + 1,5q_{sw}h_0 + 2q_1h_0 \\
&= 0,75 \times 0,75 \times 220 \times 420 + 1,5 \times 98,91 \times 420 + 2 \times 20,635 \times 420 \\
&= 131622N = 131,622kN
\end{aligned}$$

$$\begin{aligned}
Q_u(3h_0) &= 0,5R_{bt}bh_0 + 1,5q_{sw}h_0 + 3q_1h_0 \\
&= 0,5 \times 0,75 \times 220 \times 420 + 1,5 \times 98,91 \times 420 + 3 \times 20,635 \times 420 \\
&= 122963N = 122,963kN
\end{aligned}$$

$$Q_{u,1} = \min[Q_u(0); Q_u(h_0); Q_u(2h_0); Q_u(3h_0)] = 122,963kN > Q_{max} = 112,74kN$$

Hence, stirrups meet the shear bearing capacity requirements at critical sections.

Compute:

$$c_2 = \sqrt{\frac{1,5R_{bt} \times b \times h_0^2}{q_1}} = \sqrt{\frac{1,5 \times 0,75 \times 220 \times 420^2}{20,635}} = 1455mm$$

$$c_3 = \sqrt{\frac{1,5R_{bt} \times b \times h_0^2}{0,75q_{sw} + q_1}} = \sqrt{\frac{1,5 \times 0,75 \times 220 \times 420^2}{0,75 \times 98,91 + 20,635}} = 679mm$$

$c_2 = 1455mm > 2h_0 = 840mm \rightarrow$ Checking shear bearing capacity at section c_2 is not required.

$h_0 = 420mm < c_3 = 679mm < 2h_0 = 840mm$. Check the shear bearing capacity at section c_3 .

$$\begin{aligned}
Q_u(c_3) &= \sqrt{6R_{bt}bh_0^2 \times (0,75q_{sw} + q_1)} \\
&= \sqrt{6 \times 0,75 \times 220 \times 420^2 \times (0,75 \times 98,91 + 20,635)} \\
&= 128680N = 128,68kN
\end{aligned}$$

$Q_u(c_3) > Q_{max} = 112,74kN \rightarrow$ Shear bearing capacity requirements at section c_3 is satisfied.

Choose $\phi_w = 6mm$, $s = 100mm$, $n_w = 2$.

Consider the segment at the right side of support 2, $Q_{max} = Q_2^P = 91,848kN$ is used for calculation of transverse reinforcement.

The beam section is rectangular $b = 220mm$; $h = 450mm$; $h_0 = 420mm$.

Compute: $0,3R_{bt}bh_0 = 0,3 \times 8,5 \times 220 \times 420 = 235620N = 235,62kN$.

$$Q_{max} \leq 0,3R_{bt}bh_0$$

The beams are not failed by the principal compressive stress at the web!

Compute: $Q_{bmin} = 0,5R_{bt}bh_0 = 0,5 \times 0,75 \times 220 \times 420 = 34650N = 34,65kN$.

Since $Q_{max} = 91,848 > Q_{bmin}$, transverse reinforcement is required.

Pre-select $\phi_w = 6mm$, $s = 180mm$, $n_w = 2$ (2 legs). The cross-sectional area of a layer of stirrups:

$$A_{sw} = \frac{n\pi\phi_w^2}{4} = \frac{2 \times 3,14 \times 6^2}{4} = 56,52mm^2$$

$$q_{sw} = \frac{R_{sw}A_{sw}}{s} = \frac{175 \times 56,52}{180} = 54,95 \text{ N/mm}$$

$$q_{swmin} = 0,25R_{bt}b = 0,25 \times 0,75 \times 220 = 41,25 \text{ N/mm}$$

$q_{sw} > q_{swmin} \rightarrow$ Detailing requirement of stirrups is satisfied.

Compute:

$$q_1 = q_p - 0,5p_p = 31,135 - 0,5 \times 21 = 20,635 \text{ kN/m}$$

$$\begin{aligned} Q_u(0) &= 2,5R_{bt}bh_0 + 0,75q_{sw}h_0 \\ &= 2,5 \times 0,75 \times 220 \times 420 + 0,75 \times 54,95 \times 420 \\ &= 190559N = 190,559kN \end{aligned}$$

$$\begin{aligned} Q_u(h_0) &= 1,5R_{bt}bh_0 + 0,75q_{sw}h_0 + q_1h_0 \\ &= 1,5 \times 0,75 \times 220 \times 420 + 0,75 \times 54,95 \times 420 + 20,635 \times 420 \\ &= 129926N = 129,926kN \end{aligned}$$

$$\begin{aligned} Q_u(2h_0) &= 0,75R_{bt}bh_0 + 1,5q_{sw}h_0 + 2q_1h_0 \\ &= 0,75 \times 0,75 \times 220 \times 420 + 1,5 \times 54,95 \times 420 + 2 \times 20,635 \times 420 \\ &= 103927N = 103,927kN \end{aligned}$$

$$\begin{aligned} Q_u(3h_0) &= 0,5R_{bt}bh_0 + 1,5q_{sw}h_0 + 3q_1h_0 \\ &= 0,5 \times 0,75 \times 220 \times 420 + 1,5 \times 54,95 \times 420 + 3 \times 20,635 \times 420 \\ &= 95269N = 95,269kN \end{aligned}$$

$$Q_{u,1} = \min[Q_u(0); Q_u(h_0); Q_u(2h_0); Q_u(3h_0)] = 95,269kN > Q_{max} = 91,848kN$$

Hence, stirrups meet the shear bearing capacity requirements at critical sections.

Compute:

$$c_2 = \sqrt{\frac{1,5R_{bt} \times b \times h_0^2}{q_1}} = \sqrt{\frac{1,5 \times 0,75 \times 220 \times 420^2}{20,635}} = 1455mm$$

$$c_3 = \sqrt{\frac{1,5R_{bt} \times b \times h_0^2}{0,75q_{sw} + q_1}} = \sqrt{\frac{1,5 \times 0,75 \times 220 \times 420^2}{0,75 \times 54,95 + 20,635}} = 840,2mm$$

$c_2 = 1455mm > 2h_0 = 840mm \rightarrow$ Checking shear bearing capacity at section c_2 is not required.

$c_3 = 840,2mm > 2h_0 = 840mm. \rightarrow$ Checking shear bearing capacity at section c_3 is not required.

Choose $\phi_w = 6mm, s = 180mm, n_w = 2$.

At sections where shear forces are less than $Q_{bmin} = 34,65kN$, choose $\Phi 6a250$ satisfying detailing requirement. The calculation of section where shear force is equal to Q_{bmin} is based on the principle of similar triangles.

3.7. Establishing of material resistance envelope

a. Calculation of bending strength

At the end spans, there are positive bending moments and the beam cross section is in T-shaped having flange within compression zone. The effective width of the flange is $b = b_f' = 2186mm$, arrange reinforcement $2\Phi 20 + 1\Phi 20$ with cross-sectional area of $A_s = 943mm^2$. The concrete cover to the rebars is $a = 30mm$, the effective depth is $h_o = 450 - 30 = 420mm$.

$$\xi = \frac{R_s A_s}{R_b b_f' h_0} = \frac{280 \times 942}{8,5 \times 2186 \times 420} = 0,034$$

$$x = \xi \cdot h_0 = 0,034 \times 420 = 14,28mm < h_f' = 80mm - \text{the neutral axis is within flange.}$$

$$\zeta = 1 - 0,5\xi = 1 - 0,5 \times 0,034 = 0,983$$

$$M_{td} = R_s A_s \zeta h_0 = 280 \times 942 \times 0,983 \times 420 = 108,896 \times 10^6 Nmm = 108,896kNm.$$

At second support, there are negative bending moment and the beam cross section is in rectangular $b \times h = 220 \times 450$. The longitudinal rebars provided at the top of the beam are $2\Phi 18 + 1\Phi 18$ with total cross-sectional area is $A_s = 763mm^2$. The concrete cover to the center of the rebars is $a = 30mm$, the effective depth is $h_0 = 450 - 30 = 421mm$.

$$\xi = \frac{R_s A_s}{R_b b h_0} = \frac{280 \times 763}{8,5 \times 220 \times 421} = 0,271 < \xi_{pl} = 0,300$$

$$\zeta = 1 - 0,5\xi = 1 - 0,5 \times 0,271 = 0,865$$

$$M_{td} = R_s A_s \zeta h_0 = 280 \times 763 \times 0,865 \times 421 = 77,8 \times 10^6 Nmm = 77,8kNm.$$

The calculation results of bending strength are shown in *Table 3.3*. It is noted that all sections are singly - reinforced. The sections under positive moment, the width b shall be replaced by the effective width of the flange b_f' as the calculation cross section is in T-shaped. The below expressions are applied in calculation:

$$\xi = \frac{R_s A_s}{R_b b h_0}; \zeta = 1 - 0,5\xi; M_{td} = R_s A_s \zeta h_0$$

Table 3.3. Bending moment strength of sections

Sections	Quantity and cross-sectional area of provided rebars (mm²)	h_0 (mm)	ξ	ζ	M_{td} (kNm)
Mid of end span	2Φ20+1Φ20, $A_s = 942$	420	0,034	0,983	108,9
Near to support of end span	Cut 1Φ20, remain: 2Φ20, $A_s = 628$	420	0,023	0,989	73,04
Over the second support	2Φ18+1Φ18, $A_s = 763$	421	0,271	0,865	77,8
Beside the second support	Cut 1Φ18, remain: 2Φ18, $A_s = 509$	421	0,181	0,91	54,6
Mid of the second span	2Φ16+1Φ16, $A_s = 603$	422	0,022	0,989	70,47
Near to supports of the second span	Cut 1Φ16, remain: 2Φ16, $A_s = 402$	422	0,014	0,993	47,17
Over the third support	2Φ16+1Φ18, $A_s = 657$	421	0,234	0,883	68,39
Beside the third support	Cut 1Φ18, remain: 2Φ16, $A_s = 402$	422	0,143	0,929	44,13
Mid of the third span	2Φ16+1Φ16, $A_s = 603$	422	0,022	0,989	70,47
Near to the third support	Cut 1Φ16, remain: 2Φ16, $A_s = 402$	422	0,014	0,993	47,17

b. Determination of theoretical cutoff section of longitudinal rebars

The cutoff point at the right end of rebar No.4 is presented in this section. After the rebar No.4 is cut off, there are rebars No.3 (2Φ18) remaining on top at the second span, the negative moment strength is 54,6 kNm. The material resistance envelope intersects with the bending moment envelope at point H. This is the theoretical cutoff section of rebar No.4. The distance from point H to the right edge of the second support is determined that based on the principle of similar triangles to be 606mm (Figure 3.3).

Determination of extension length W_4^p based on the principle of similar triangles and the corresponding shear force at point H is $Q = 72,97kN$.

In this beam segment, the stirrups provided are Φ6a180.

$$q_{sw} = \frac{R_{sw} A_{sw}}{s} = \frac{175 \times 56,52}{180} = 54,95 N / mm = 54,95 kN / m$$

There is inclined rebars in this area, then $Q_{s,inc} = 0$.

Hence:

$$\begin{aligned} W_4^p &= \frac{Q - Q_{s,inc}}{2q_{sw}} + 5\phi \\ &= \frac{72,97 - 0}{2 \times 54,95} + 5 \times 0,018 = 0,754m > 20\phi = 20 \times 0,018 = 0,36m \end{aligned}$$

Choose $W_4^p = 760mm$. The actual cutoff point is at a distance of $606 + 760 = 1366mm$ from the right edge of the second support and of $1366 + 150 = 1516mm$ from axis 2.

Similar calculations are conducted for cutoff points of the other rebars. The results are shown in *Table 3.4*.

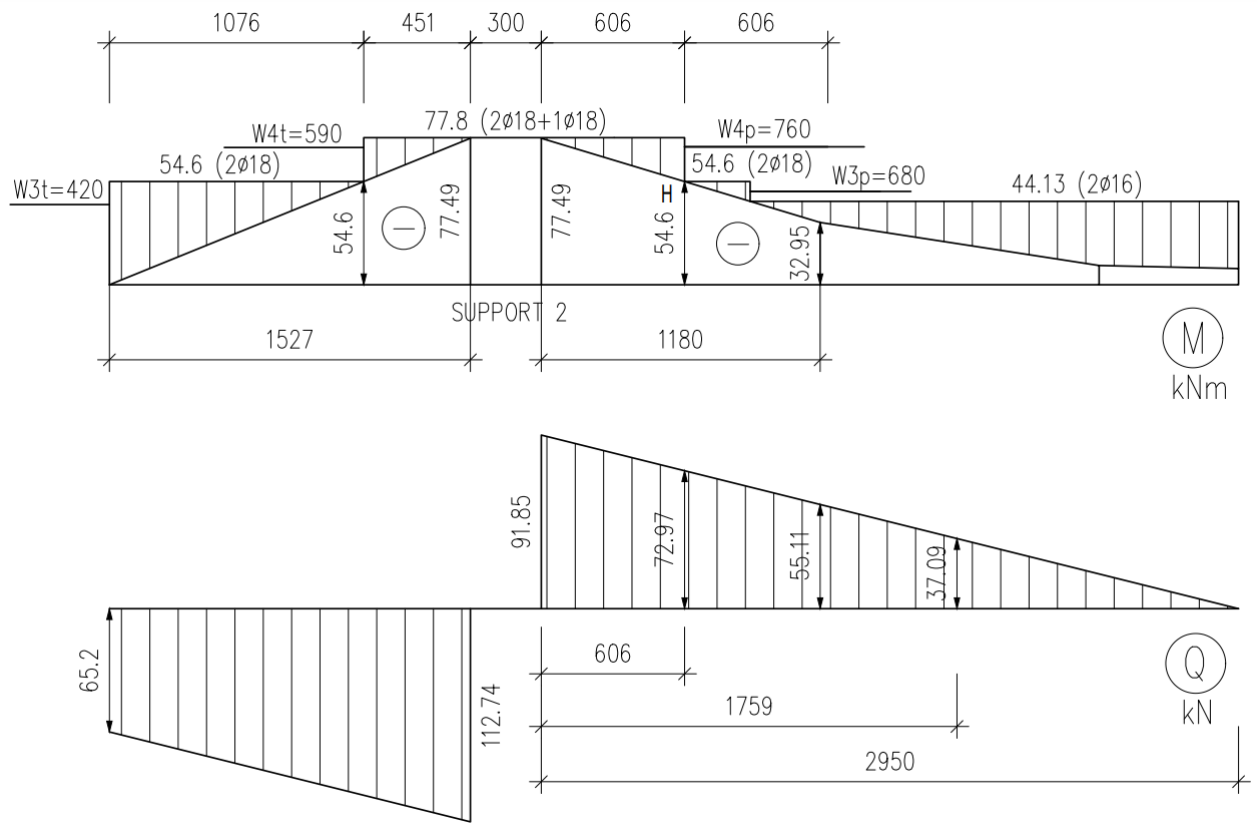


Figure 3.3. Theoretical cutoff section of rebar No.4

Similar calculations are conducted for cutoff points of the other rebars. The results are shown in *Table 3.4*.

Table 3.4. Theoretical cutoff sections of longitudinal rebars

Rebars	Theoretical cutoff sections	Extension length
No.2 (left end)	At a distance of 1191mm from pin 1	$W_2^t = 489mm$
No.2 (right end)	2103mm from left edge of pin 2	$W_2^p = 410mm$
No.3 (left end)	1527mm from left edge of pin 2	$W_3^t = 420mm$
No.3 (right end)	884mm from right edge of pin 2	$W_3^p = 679mm$
No.4 (left end)	451mm from left edge of pin 2	$W_4^t = 593mm$
No.6 (left end)	1759mm from right edge of pin 2	$W_6^t = 419mm$
No.6 (right end)	1759mm from left edge of pin 3	$W_6^p = 419mm$
No.8 (left end)	675mm from left edge of pin 3	$W_8^t = 739mm$
No.8 (right end)	656mm from right edge of pin 3	$W_8^p = 740mm$

3.8. Anchorage of longitudinal rebars

The remaining bottom rebars anchored into support shall be at least 1/3 in area compared to that of the longitudinal rebars at mid-span.

- End spans: $2\Phi 20 + 1\Phi 20$, cut $1\Phi 20$ and remain $2\Phi 20$, anchored area of 66,7%.
- Interior spans: $2\Phi 16 + 1\Phi 16$, cut $1\Phi 16$ and remain $2\Phi 16$, anchored area of 66,7%.

At support 2:

$$Q_{max} = 112,74 < Q_{b,1} = 2,5R_{bt}bh_0 = 2,5 \times 0,75 \times 220 \times 420 = 173250kN$$

Calculate:

$$0,3L_{0,an} = 0,3\phi(R_s / R_{bt}) / (4\eta_1\eta_2) = 0,3 \times \phi \times (280 / 0,75) / (4 \times 2,5 \times 1) = 11,2\phi$$

$$\text{Hence: } l_{an} \geq \max(15\phi; 200; 0,3L_{0,an}) = \max(300; 200; 230) = 300mm.$$

3.9. Constructive reinforcement

Rebars No.9 ($2\Phi 12$) having $A_s = 226mm^2 > 0,1\%bh_0 = 0,001 \times 220 \times 420 = 92mm^2$. These contribution rebars are used for top side at end span, where there is no negative moment.

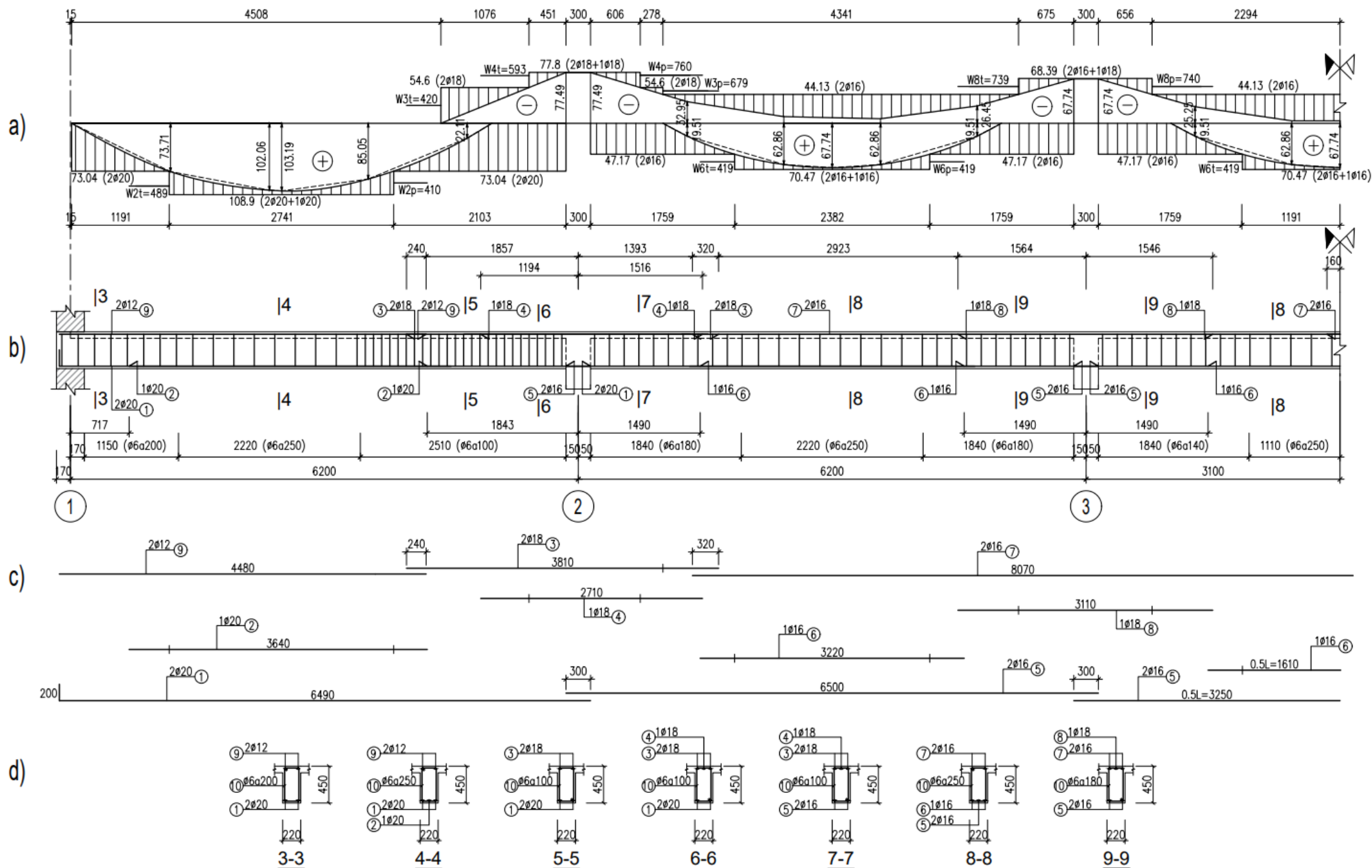


Figure 3.4. Reinforcement arrangement of secondary beams
a) Material resistance envelope; b) Section; c) Rebar development; d) Cross sections

IV. DESIGN OF PRIMARY BEAMS

4.1. Calculation diagram

Primary beams are four-span continuous beam with cross-section sizes of $h_{dc} = 700mm$; $b_{dc} = 300mm$. They are supported by columns with $b_c = h_c = 300mm$. The design span length is $l = 7,05m$ for both end spans and interior spans.

As the primary beams are symmetrical, the left-side half of the beams including two spans will be considered. The calculation diagram is shown in Figure 4.1.

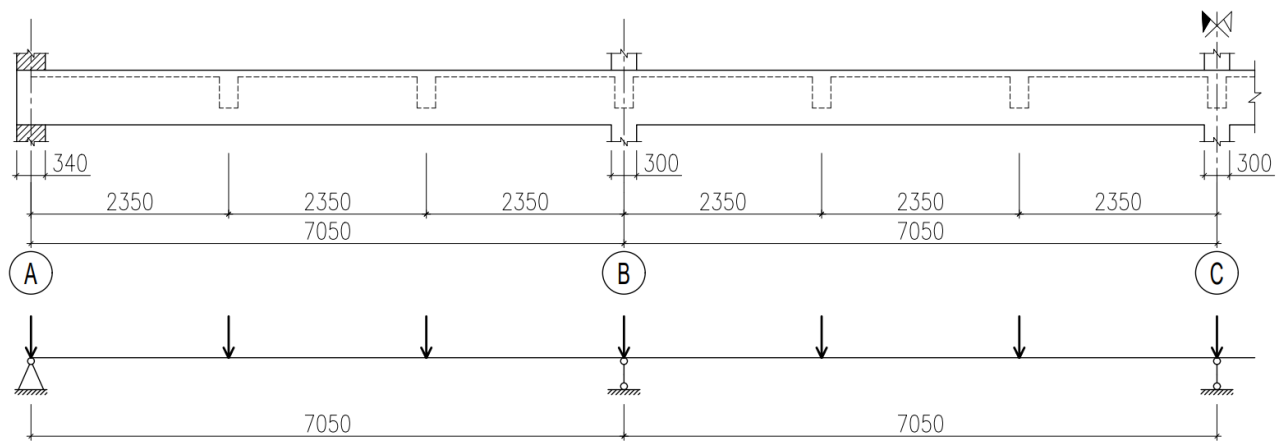


Figure 4.1. Calculation diagram of primary beams

4.2. Design loads

Self-weight of beam is assumed to act as an equivalent point load. The slab thickness shall be subtracted since it is already considered in the slab calculation:

$$G_0 = b_{dc}(h_{dc} - h_b)\gamma_n l_1 = 0,30 \times (0,65 - 0,08) \times 25 \times 1,1 \times 2,35 = 11,051 kN$$

Dead load transmitted from secondary beam:

$$G_1 = g_{dp} l_2 = 10,135 \times 6,2 = 62,837 kN$$

Concentrated dead load:

$$G = G_0 + G_1 = 11,051 + 62,837 = 73,888 kN$$

Concentrated live load transmitted from secondary beam:

$$P = p_{dp} l_2 = 21 \times 6,2 = 130,2 kN$$

4.3. Design internal forces:

a. Determination the envelope diagram of bending moment:

In this case, internal forces in primary beams are determined from elastic analysis.

The unfavorable load cases shall be determined (Figure 4.3).

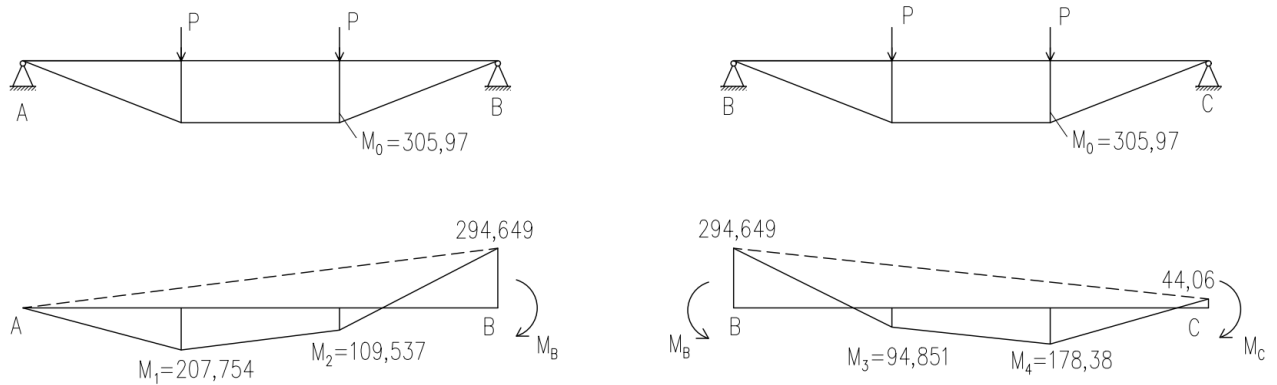


Figure 4.2. Diagram for calculation of bending moment at some sections

Bending moment diagram of dead load G :

$$M_G = \alpha Gl = \alpha \times 73,888 \times 7,05 = \alpha \times 520,91 \text{ (kNm)}$$

The coefficient α is taken from *Appendix 12* in “*Design of cast-in-situ reinforced concrete slab and beam structures*”.

Bending moment diagrams of live load P_i :

$$M_{Pi} = \alpha Pl = \alpha \times 130,2 \times 7,05 = \alpha \times 917,91 \text{ (kNm)}.$$

The six unfavorable cases of live load are shown in *Figure 4.3* (c, d, e, f, g, h).

In the diagram of M_{P3} , the coefficient α for section 1,2,3 and 4 are not given in the appendix. Spans AB and BC are separated for calculation. As there are loads on spans 1 and 2, the bending moment M_0 of simply-supported beams is $M_0 = Pl_1 = 130,2 \times 2,35 = 305,97 \text{ kNm}$. Based on the principle of similar triangles and hanging diagram method, the following values of bending moment can be determined

$$M_1 = 305,97 - 294,649 \times (1/3) = 207,754 \text{ kNm}$$

$$M_2 = 305,97 - 294,649 \times (2/3) = 109,537 \text{ kNm}$$

$$M_3 = 305,97 - (294,649 - 44,06) \times (2/3) - 44,06 = 94,851 \text{ kNm}$$

$$M_4 = 305,97 - (294,649 - 44,06) \times (1/3) - 44,06 = 178,38 \text{ kNm}$$

The calculated results are shown in Table 4.1.

Table 4.1. Calculation and combination of bending moment

Moment (kNm)		1	2	B	3	4	C
M_G	α	0,238	0,143	-0,286	0,079	0,111	-0,190
	M	123,977	74,49	-148,98	41,152	57,821	-98,973
M_{P1}	α	0,286	0,238	-0,143	-0,127	-0,111	-0,095
	M	262,522	218,463	-131,261	-116,575	-101,888	-87,201
M_{P2}	α	-0,048	-0,095	-0,143	0,206	0,222	-0,095
	M	-44,06	-87,201	-131,261	189,089	203,776	-87,201
M_{P3}	α			-0,321			-0,048
	M	207,754	109,537	-294,649	94,851	178,38	-44,06
M_{P4}	α	-0,031	-0,063	-0,095			-0,286
	M	-28,455	-57,828	-87,201	160,329	101,888	-262,522
M_{P5}	α			-0,190			0,095
	M	247,836	189,701	-174,403	-87,201	0	87,201
M_{P6}	α			0,036			-0,143
	M	11,015	22,03	33,045	-21,724	-76,492	-131,261
M_{max}		386,499	292,953	-115,935	230,241	261,597	-11,772
M_{min}		79,917	-12,711	-443,629	-75,423	-44,067	-361,495

The ordinates of the peak point of bending moment envelope:

$$M_{max} = M_G + \max (M_{Pi}); M_{min} = M_G + \min (M_{Pi})$$

The values of M_{max} and M_{min} are calculated for each section and are given in the last line of Table 4.1.

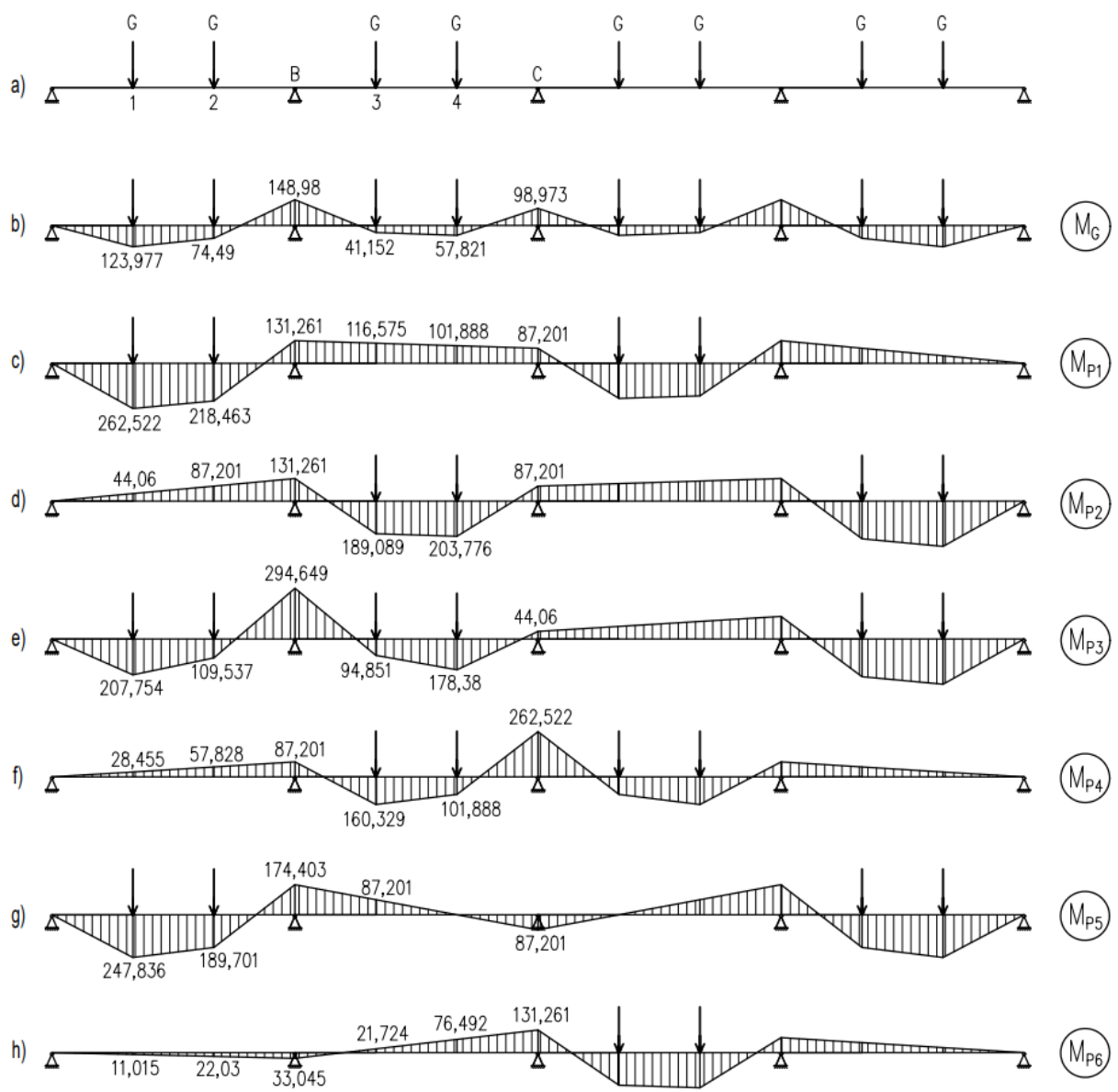


Figure 4.3. Diagram of bending moment of load cases

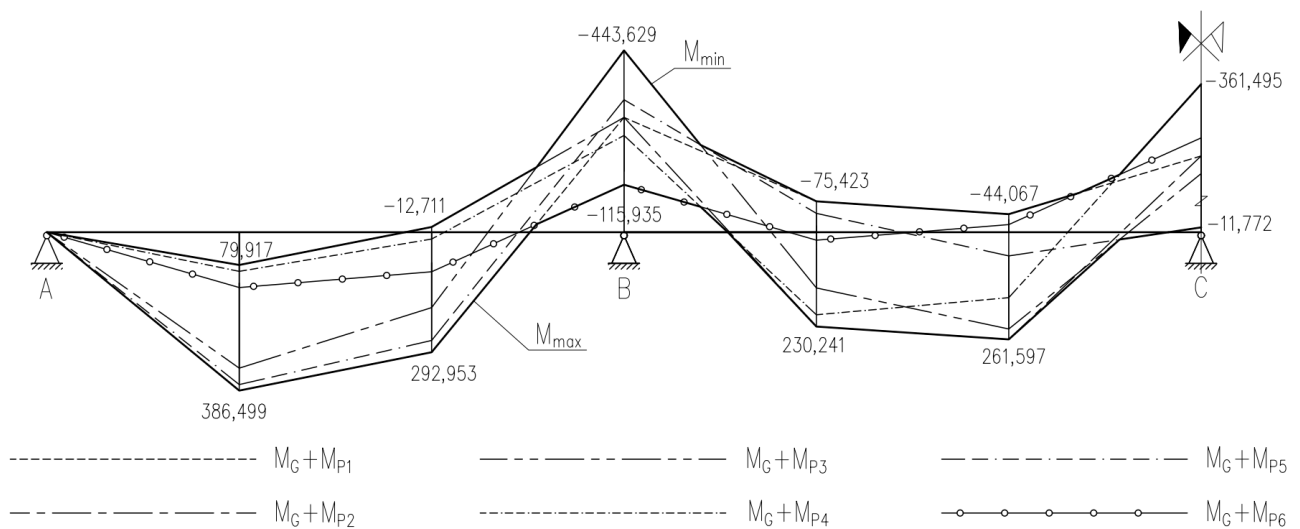


Figure 4.4. Bending moment envelope obtained from the combination method

The bending moments at the support edges can be determined from the bending moment envelope. At the right-hand side of support B, the slope of the negative moment diagram is higher than that at the left-hand side. Hence, the absolute value of negative moment at the right-hand side of the support will be more dominant.

$$M_{mg} = M_g - \frac{h_c(M_g + M_3)}{2l_1}$$

$$M_3 = 41,152 + 94,851 = 136,003 \text{ kNm}$$

$$M_{mg}^B = 443,629 - \frac{0,3 \times (443,629 + 136,003)}{2 \times 2,35} = 406,631 \text{ kNm}$$

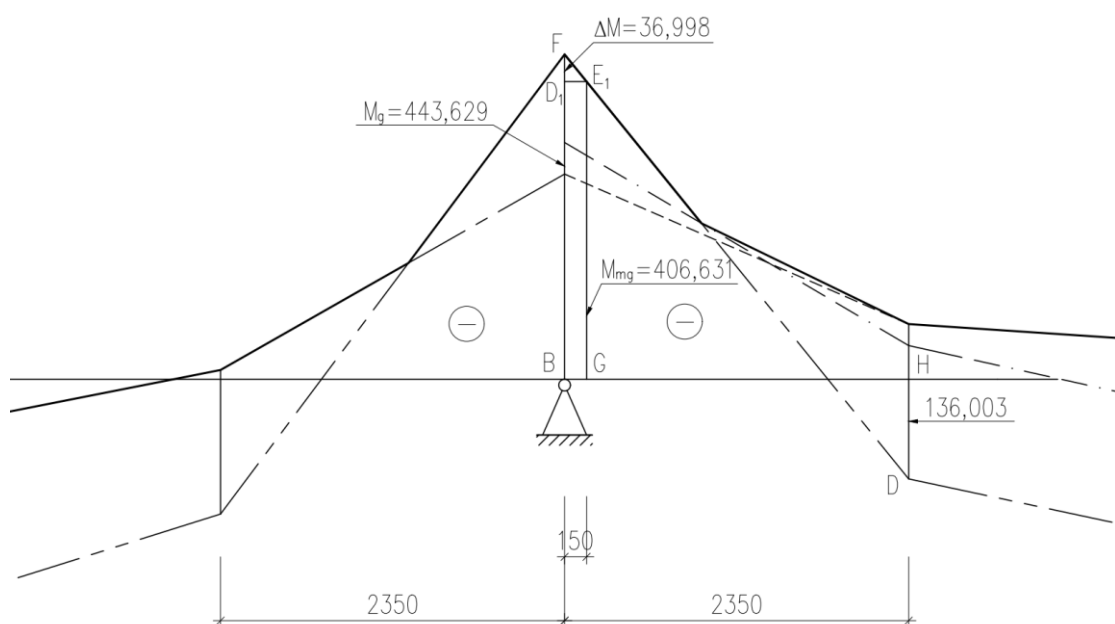


Figure 4.5. Calculation diagram to determine M_{mg}

The bending moment at the edge of support C is determined similarly: $M_{mg}^C = 328,227kNm$.

b. Determination of shear forces envelope:

The ordinates of shear forces envelope are:

- Due to dead loads G : $Q_G = \beta G = \beta \times 73,888 \text{ (kN)}$
- Due to live loads P_i : $Q_{P_i} = \beta_i P = \beta_i \times 130,2 \text{ (kN)}$

The coefficient β is taken from *Appendix 12* in “*Design of cast-in-situ reinforced concrete slab and beam structures*”, the associated load cases are shown in *Figure 4.3*, the result obtained are listed in *Table 4.2*.

Table 4.2. Caculation and Combination of the Shear Forces

Shear force (kN)		Right of support A	Middle of the outer span	Left of support B	Right of support B	Middle of Span 2	Left of support C
Q_G	β	0,714	...	-1,286	1,005	...	-0,995
	Q	52,756	-21,132	-95,02	74,257	0,369	-73,519
Q_{P1}	β	0,857	...	-1,143	0,048	...	0
	Q	111,581	-18,619	-148,819	6,25	0	0
Q_{P2}	β	-0,143	...	-0,143	1.048	...	-0,952
	Q	-18,619	-18,619	-18,619	136,45	6,25	-123,95
Q_{P3}	β	0,679	...	-1,321	1.274	...	-0,726
	Q	88,406	-41,794	-171,994	165,875	35,675	-94,525
Q_{P4}	β	-0,095		-0,095	0,810	...	-1,190
	Q	-12,369	-12,369	-12,369	105,462	-24,738	-154,938
Q_{P5}	β	0,810	...	-1,190	0,286	...	0,286
	Q	105,462	-24,738	-154,938	37,237	37,237	37,237
Q_{P6}	β	0	0	0,036	0,187	...	...
	Q	0	0	4,687	24,347		
Q_{max}		164,337	-33,501	-90,333	240,132	37,606	-36,282
Q_{min}		34,137	-62,926	-267,014	80,507	-24,369	-228,457

In the mid-span, the shear forces can be determined based on the section method from the equilibrium of the free body of a beam segment. For example, at the middle of the end span:

$$Q = Q_A - G = 52,756 - 73,888 = -21,132\text{kN}.$$

Normally, the shear forces at mid-span are quite small. Hence, transverse reinforcement at this area will follow the detailing requirement.

The envelope diagram for shear forces is shown in *Figure 4.6*.

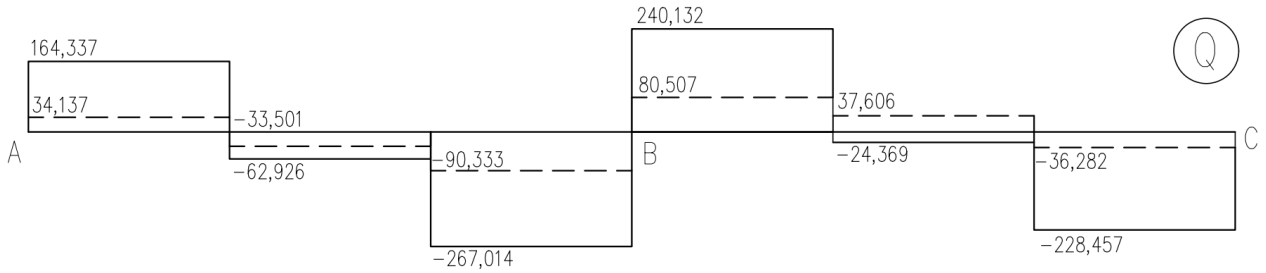


Figure 4.6. Shear forces envelope

4.4. Calculation of reinforcement for bending (longitudinal rebars)

Concrete compressive strength class B15 with $R_b = 8,5\text{MPa}$; $R_{bt} = 0,9\text{MPa}$; CII class reinforcement with $R_s = 280\text{MPa}$, $R_{sc} = 280\text{MPa}$ for longitudinal rebars.

Primary beams are analysed by elastic theorem with the limit relative depth of compression zone $\zeta_R = 0,65$ and $\alpha_R = 0,439$ (based on Appendix 9 in “*Design of cast-in-situ reinforced concrete slab and beam structures*” with the partial performance coefficient of concrete $\gamma_{b2} = 1,0$).

a. For negative bending moments

The beam cross section is calculated in rectangular $b = 300\text{mm}$, $h = 700\text{mm}$.

Since at the beam intersection, the top longitudinal rebars of primary beams are below those of secondary beams, it is assumed that $a = 70\text{mm}$, $h_0 = 700 - 70 = 630\text{mm}$.

At support B, with $M_{mg} = 406,631\text{kNm}$

$$\alpha_m = \frac{M}{R_b b h_0^2} = \frac{406,631 \times 10^6}{8,5 \times 300 \times 630^2} = 0,402 < \alpha_R = 0,439$$

$$\zeta = \frac{1 + \sqrt{1 - 2\alpha_m}}{2} = \frac{1 + \sqrt{1 - 2 \times 0,402}}{2} = 0,721$$

$$A_s = \frac{M}{R_s \zeta h_0} = \frac{406,631 \times 10^6}{280 \times 0,721 \times 630} = 3198\text{mm}^2$$

Check $\mu\% = \frac{A_s}{b_{dc} h_0} = \frac{3198}{300 \times 630} \times 100 = 1,69\%$

Similarly, at support C, with $M_{mg} = 328,227\text{kNm}$; $\alpha_m = 0,324$; $\zeta = 0,797$; $A_s = 2335\text{mm}^2$, $\mu = 1,24\%$.

b. For positive bending moments

The beam cross section is calculated in T-shaped having flange in compression zone, the flange thickness $h_f = 80mm$.

Assume that $a = 45mm$, $h_0 = 700 - 45 = 655mm$.

The overhang length of the flange does not exceed the maximum value of:

$$- (1/6)l_d = (1/6) \times 7,05 = 1,175m$$

- A half of clear distance between two adjacent primary beams: $0,5l = 0,5 \times (6,2 - 0,3) = 2,95m$ (due to $h_f > 0,1h$; with $h = 700mm$ and transverse beams are secondary beams with distance of $2,15m$).

Then $S_f \leq \min(1,175; 2,95) = 1,175m$. Choose $S_f = 1,175m = 1175mm$.

The effective width of flange:

$$b_f = b + 2S_f = 300 + 2 \times 1175 = 2650mm.$$

Compute:

$$M_f = R_b b_f h_f (h_0 - 0,5h_f) = 8,5 \times 2650 \times 80 \times (655 - 0,5 \times 80) = 1108,23 \times 10^6 Nmm.$$

Then $M_{max} = 386,499kNm < M_f = 1108,23kNm \rightarrow$ Neutral axis is within the flange.

Calculation in rectangular $b = b_f = 2650mm$, $h = 700mm$, $a = 45mm$, $h_0 = 655mm$.

At mid-span of end spans, with $M = 386,499kNm$

$$\alpha_m = \frac{M}{R_b b_f h_0^2} = \frac{386,499 \times 10^6}{8,5 \times 2650 \times 655^2} = 0,04 < \alpha_R = 0,439$$

$$\zeta = \frac{1 + \sqrt{1 - 2\alpha_m}}{2} = \frac{1 + \sqrt{1 - 2 \times 0,04}}{2} = 0,980$$

$$A_s = \frac{M}{R_s \zeta h_0} = \frac{386,499 \times 10^6}{280 \times 0,980 \times 655} = 2151mm^2$$

Check

$$\mu\% = \frac{A_s}{b_{dc} h_0} = \frac{2151}{300 \times 655} \times 100 = 1,09\%$$

At second span, $M = 261,597kNm$

$$\alpha_m = 0,027; \zeta = 0,986; A_s = 1447mm^2; \mu = 0,74\%.$$

Longitudinal rebars are selected and arranged as follows.

The required and provided longitudinal rebars are shown in Table 4.3.

Table 4.3. Calculation and selection of longitudinal reinforcement for primary beam

Section	End span	Support B	Span 2	Support C
A_s required	2151mm^2	3198mm^2	1447mm^2	2335mm^2
Rebars provided	$2\Phi 25+2\Phi 28$	$4\Phi 28+2\Phi 22$	$2\Phi 25+1\Phi 25$	$2\Phi 28+2\Phi 28$
A_s provided	2213mm^2	3223mm^2	1473mm^2	2463mm^2

The longitudinal rebars at main sections are arranged as shown in *Figure 4.7*.

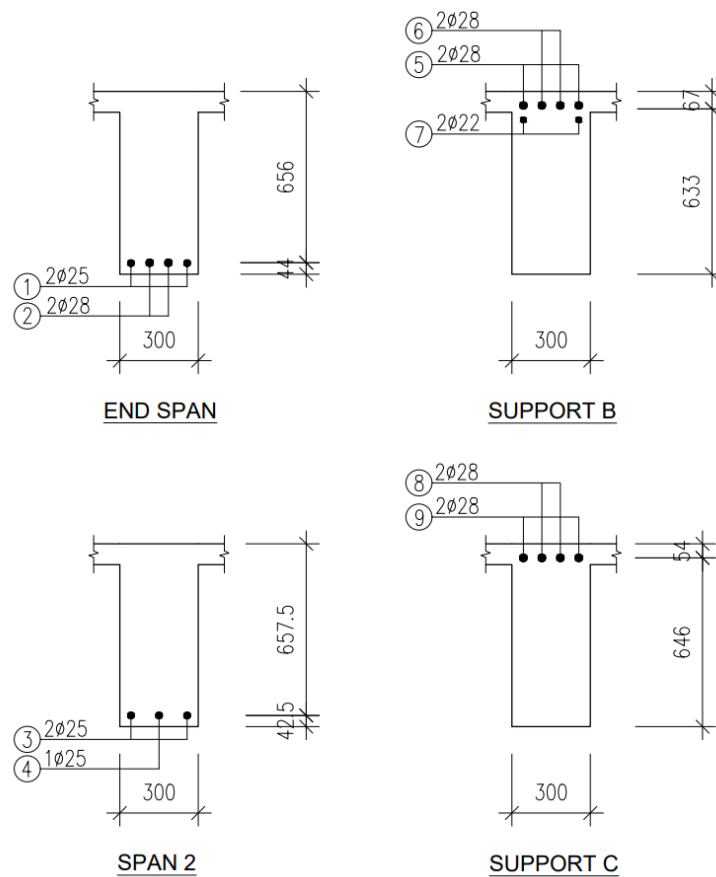


Figure 4.7. Longitudinal reinforcement arrangement at the main section

Check for the actual effective depth h_0 :

With the rebars for positive bending moments, the concrete cover to the outer edges of rebars is $c = 30\text{mm}$, that is greater than $\Phi_{\max} = 28\text{mm}$.

$$a = c + 0,5\Phi_{\max} = 30 + 0,5 \times 28 = 44\text{mm}$$

Hence, the actual effective depth is $h_{0t} = h - a = 700 - 44 = 656\text{mm}$. The pre-assumed effective depth of 655mm is conservative.

With the rebars for negative bending moments, the concrete cover to outer edges of $\Phi 20$ rebars of the secondary beams is $c_s=20mm$. Hence, the concrete cover to the outer edges of $\Phi 28$ rebars of the main beams is $c_m = 20 + 20 = 40mm$.

At support C, there is one layer of top rebar in the main beam, the actual concrete cover is $a = c_m + 0,5\Phi_{\max} = 40 + 0,5 \times 28 = 54mm$, smaller than the pre-assumed value, which is conservative.

At support B, there are two layers of top rebars in main beams, the outer layer includes $4\Phi 28$ ($A_{s1} = 2463mm^2$, $a_1 = c_m + 0,5\Phi_{\max} = 40 + 0,5 \times 28 = 54mm$), the interior layer includes $2\Phi 22$ with the clear distance to the outer layer of $t = 30mm$ ($A_{s2} = 760mm^2$, $a_2 = c_m + \Phi_{\max} + t + 0,5\Phi = 40 + 28 + 30 + 0,5 \times 22 = 109mm$). The distance from the nearest concrete surface to the centroid of $4\Phi 28+2\Phi 22$ is:

$$a = \frac{a_1 A_{s1} + a_2 A_{s2}}{A_{s1} + A_{s2}} = \frac{54 \times 2463 + 109 \times 760}{2463 + 760} = 67mm$$

The actual effective depth is $h_0 = h - a = 700 - 67 = 633mm$. The pre-assumed effective depth of $630mm$ is conservative.

Check the clear distance between the longitudinal rebars:

The concrete cover to the outer edges of longitudinal rebars is $30mm$, then in the first layer of $4\Phi 28$, the equal clear distance between the bars is:

$$t = \frac{b - 2c - 4\Phi}{3} = \frac{300 - 2 \times 30 - 4 \times 28}{3} = 43mm > 30mm$$

4.5. Calculation of reinforcement for shear

Concrete compressive strength class B15 with $R_b = 8,5MPa$, $R_{bt} = 0,75MPa$; C-I class reinforcement with $R_{sw} = 175MPa$ for transverse rebars.

The shear force envelope in Figure 4.6 shows that:

- At the right side of support A, the shear force $Q_A^P = 164,337kN$ is constant in l_1 .
- At the left side of support B, the shear force $Q_B^T = 267,014kN$ is constant in l_1 .
- At the right side of support B, the shear force $Q_B^P = 240,132kN$ is constant in l_1 .
- At the left side of support C, the shear force $Q_C^T = 228,457kN$ is constant in l_1 .

Calculation of stirrups (without inclined rebars):

Consider the l_1 - length segment at the right side of support A, $Q_A^P = 164,337kN$.

Beam cross section: $b = 300\text{mm}$, $h = 700\text{mm}$, $h_0 = 633\text{mm}$.

(The lowest effective depth $h_0 = 633\text{mm}$ is used which means that designs will be safer).

Compute: $Q_{bmin} = 0,5R_{bt}bh_0 = 0,5 \times 0,75 \times 300 \times 633 = 71212,5\text{N} = 71,213\text{kN}$

Since $Q_{max} = 164,337 > Q_{bmin}$, transverse reinforcement is required.

The condition that concrete in the strips between inclined cracks at the beam web is not crushed due to principal compressive stresses is checked:

$$Q_{max} = 164,337\text{kN} < 0,3R_bbh_0 = 0,3 \times 8,5 \times 300 \times 633 = 484245\text{N} = 484,245\text{kN}$$

Hence, stirrups shall be designed for shear.

Pre-select $\phi_w = 8\text{mm}$, $n_w = 2$, $s = 170\text{mm}$. The cross-sectional area of a layer of stirrups is:

$$A_{sw} = \frac{n\pi\phi_w^2}{4} = \frac{2 \times 3,14 \times 8^2}{4} = 100,48\text{mm}^2$$

$$q_{sw} = \frac{R_{sw}A_{sw}}{s} = \frac{175 \times 100,48}{170} = 103,44\text{N/mm}$$

$$q_{swmin} = 0,25R_{bt}b = 0,25 \times 0,75 \times 300 = 56,25\text{N/mm}$$

$$q_{sw} > q_{swmin} \rightarrow \text{selected stirrups satisfying detailing requirement.}$$

Compute:

$$c_1 = \sqrt{\frac{1,5R_{bt}bh_0^2}{0,75q_{sw}}} = \sqrt{\frac{1,5 \times 0,75 \times 300 \times 633^2}{0,75 \times 103,44}} = 1320,3\text{mm}$$

Distance from support to nearest concentrated load $a = 2350\text{mm}$.

Consider case: $3h_0 \leq a$ and $2h_0 < c_1$

$$\begin{aligned} Q_u(3h_0) &= 0,5R_{bt}bh_0 + 0,75q_{sw}2h_0 \\ &= 0,5 \times 0,75 \times 300 \times 633 + 0,75 \times 103,44 \times 2 \times 633 \\ &= 169429\text{N} = 169,429\text{kN} > Q_{max} = 164,337\text{kN} \end{aligned}$$

Choose $\phi_w = 8\text{mm}$, $n_w = 2$, $s = 170\text{mm}$ for stirrups.

Consider the l_1 - length segment at the left side of support B, $Q_B^T = 267,014\text{kN}$.

Beam cross section: $b = 300\text{mm}$, $h = 700\text{mm}$, $h_0 = 633\text{mm}$.

Compute: $Q_{bmin} = 0,5R_{bt}bh_0 = 0,5 \times 0,75 \times 300 \times 633 = 71212,5\text{N} = 71,213\text{kN}$

Since $Q_{max} = 267,014 > Q_{bmin}$, transverse reinforcement is required.

The condition that concrete in the strips between inclined cracks at the beam web is not crushed due to principal compressive stresses is checked:

$$Q_{max} = 267,014kN < 0,3R_b b h_0 = 0,3 \times 8,5 \times 300 \times 633 = 484245N = 484,245kN$$

Hence, stirrups shall be designed for shear.

Pre-select $\phi_w = 10mm$, $n_w = 2$, $s = 120mm$. The cross- sectional area of a layer of stirrups is:

$$A_{sw} = \frac{n\pi\phi_w^2}{4} = \frac{2 \times 3,14 \times 10^2}{4} = 157mm^2$$

$$q_{sw} = \frac{R_{sw} A_{sw}}{s} = \frac{175 \times 157}{120} = 228,96 N/mm$$

$$q_{swmin} = 0,25R_{bt}b = 0,25 \times 0,75 \times 300 = 56,25 N/mm$$

$q_{sw} > q_{swmin} \rightarrow$ selected stirrups satisfying detailing requirement.

Compute:

$$c_1 = \sqrt{\frac{1,5R_{bt}bh_0^2}{0,75q_{sw}}} = \sqrt{\frac{1,5 \times 0,75 \times 300 \times 633^2}{0,75 \times 228,96}} = 887,4mm$$

Distance from support to nearest concentrated load $a = 2350mm$.

Consider case: $3h_0 \leq a$ and $h_0 < c_1 < 2h_0$

$$\begin{aligned} Q_u(c_1) &= 2\sqrt{0,75 \times 1,5R_{bt}bh_0^2q_{sw}} \\ &= 2\sqrt{0,75 \times 1,5 \times 0,75 \times 300 \times 633^2 \times 228,96} \\ &= 304776N = 304,776kN \end{aligned}$$

$$\begin{aligned} Q_u(3h_0) &= 0,5R_{bt}bh_0 + 0,75q_{sw}2h_0 \\ &= 0,5 \times 0,75 \times 300 \times 633 + 0,75 \times 228,96 \times 2 \times 633 \\ &= 288610N = 288,61kN \end{aligned}$$

$$Q_u = \min[Q_u(c_1); Q_u(3h_0)] = 288,61kN > Q_{max} = 267,014kN$$

Choose $\phi_w = 10mm$, $n_w = 2$, $s = 120mm$ for stirrups.

Consider the l_1 - length segment at the right side of support B, $Q_B^P = 240,132kN$.

Beam cross section: $b = 300mm$, $h = 700mm$, $h_0 = 633mm$.

Compute: $Q_{bmin} = 0,5R_{bt}bh_0 = 0,5 \times 0,75 \times 300 \times 633 = 71212,5N = 71,213kN$

Since $Q_{max} = 240,132 > Q_{bmin}$, transverse reinforcement is required.

The condition that concrete in the strips between inclined cracks at the beam web is not crushed due to principal compressive stresses is checked:

$$Q_{max} = 240,132kN < 0,3R_b b h_0 = 0,3 \times 8,5 \times 300 \times 633 = 484245N = 484,245kN$$

Hence, stirrups shall be designed for shear.

Pre-select $\phi_w = 10mm$, $n_w = 2$, $s = 140mm$. The cross- sectional area of a layer of stirrups is:

$$A_{sw} = \frac{n\pi\phi_w^2}{4} = \frac{2 \times 3,14 \times 10^2}{4} = 157mm^2$$

$$q_{sw} = \frac{R_{sw} A_{sw}}{s} = \frac{175 \times 157}{140} = 196,25 N/mm$$

$$q_{swmin} = 0,25R_{bt}b = 0,25 \times 0,75 \times 300 = 56,25 N/mm$$

$q_{sw} > q_{swmin} \rightarrow$ selected stirrups satisfying detailing requirement.

Compute:

$$c_1 = \sqrt{\frac{1,5R_{bt}bh_0^2}{0,75q_{sw}}} = \sqrt{\frac{1,5 \times 0,75 \times 300 \times 633^2}{0,75 \times 196,25}} = 958,5mm$$

Distance from support to nearest concentrated load $a = 2350mm$.

Consider case: $3h_0 \leq a$ and $h_0 < c_1 < 2h_0$

$$\begin{aligned} Q_u(c_1) &= 2\sqrt{0,75 \times 1,5R_{bt}bh_0^2q_{sw}} \\ &= 2\sqrt{0,75 \times 1,5 \times 0,75 \times 300 \times 633^2 \times 196,25} \\ &= 282167N = 282,167kN \end{aligned}$$

$$\begin{aligned} Q_u(3h_0) &= 0,5R_{bt}bh_0 + 0,75q_{sw}2h_0 \\ &= 0,5 \times 0,75 \times 300 \times 633 + 0,75 \times 196,25 \times 2 \times 633 \\ &= 257552N = 257,552kN \end{aligned}$$

$$Q_u = \min[Q_u(c_1); Q_u(3h_0)] = 257,552kN > Q_{max} = 240,132kN$$

Choose $\phi_w = 10mm$, $n_w = 2$, $s = 140mm$ for stirrups.

Consider the l_1 - length segment at the left side of support C, $Q_C^T = 228,457kN$.

Beam cross section: $b = 300mm$, $h = 700mm$, $h_0 = 633mm$.

(The lowest effective depth $h_0 = 633mm$ is used which means that design will be safer).

Compute: $Q_{bmin} = 0,5R_{bt}bh_0 = 0,5 \times 0,75 \times 300 \times 633 = 71212,5N = 71,213kN$

Since $Q_{max} = 228,457 > Q_{bmin}$, transverse reinforcement is required.

The condition that concrete in the strips between inclined cracks at the beam web is not crushed due to principal compressive stresses is checked:

$$Q_{max} = 228,457kN < 0,3R_bbh_0 = 0,3 \times 8,5 \times 300 \times 633 = 484245N = 484,245kN$$

Hence, stirrups shall be designed for shear.

Pre-select $\phi_w = 10mm$, $n_w = 2$, $s = 160mm$. The cross- sectional area of a layer of stirrups is:

$$A_{sw} = \frac{n\pi\phi_w^2}{4} = \frac{2 \times 3,14 \times 10^2}{4} = 157 \text{ mm}^2$$

$$q_{sw} = \frac{R_{sw} A_{sw}}{s} = \frac{175 \times 157}{160} = 171,72 \text{ N/mm}$$

$$q_{swmin} = 0,25 R_{bt} b = 0,25 \times 0,75 \times 300 = 56,25 \text{ N/mm}$$

$q_{sw} > q_{swmin} \rightarrow$ selected stirrups satisfying detailing requirement.

Compute:

$$c_1 = \sqrt{\frac{1,5 R_{bt} b h_0^2}{0,75 q_{sw}}} = \sqrt{\frac{1,5 \times 0,75 \times 300 \times 633^2}{0,75 \times 171,72}} = 1024,7 \text{ mm}$$

Distance from support to nearest concentrated load $a = 2350 \text{ mm}$.

Consider case: $3h_0 \leq a$ and $h_0 < c_1 < 2h_0$

$$\begin{aligned} Q_u(c_1) &= 2\sqrt{0,75 \times 1,5 R_{bt} b h_0^2 q_{sw}} \\ &= 2\sqrt{0,75 \times 1,5 \times 0,75 \times 300 \times 633^2 \times 171,72} \\ &= 263944 \text{ N} = 263,944 \text{ kN} \end{aligned}$$

$$\begin{aligned} Q_u(3h_0) &= 0,5 R_{bt} b h_0 + 0,75 q_{sw} 2h_0 \\ &= 0,5 \times 0,75 \times 300 \times 633 + 0,75 \times 171,72 \times 2 \times 633 \\ &= 234261 \text{ N} = 234,261 \text{ kN} \end{aligned}$$

$$Q_u = \min[Q_u(c_1); Q_u(3h_0)] = 234,261 \text{ kN} > Q_{max} = 228,457 \text{ kN}$$

Choose $\phi_w = 10 \text{ mm}$, $n_w = 2$, $s = 160 \text{ mm}$ for stirrups.

For the remaining beam segments where shear forces are less than $Q_{bmin} = 71,213 \text{ kN}$, choose $\Phi 8 \text{ a } 250$ satisfying detailing requirement.

4.6. Calculation of reinforcement for breaking-off:

Hanger stirrups are required to strengthen the primary beams where they support secondary beams. The concentrated load transmitted from the secondary beams to the primary beam is:

$$P_1 = P + G_1 = 130,2 + 62,837 = 193,037kN.$$

The required area of hanger stirrups for breaking-off strength is:

$$A_{sw} = \frac{P_1 \left(1 - \frac{h_s}{h_0}\right)}{R_{sw}} = \frac{193,037 \times 10^3 \times \left(1 - \frac{206}{656}\right)}{175} = 757mm^2$$

$$\text{where: } h_s = h_o^{dc} - h_{dp} = 656 - 450 = 206mm.$$

Select $\phi = 10$, and $n_s = 2$ (2 legs). The cross-sectional area of a leg of stirrups is:

$$a_s = \frac{\pi \phi^2}{4} = \frac{3,14 \times 10^2}{4} = 78,5mm$$

The required number of hanger stirrups is:

$$m = \frac{A_{sw}}{n_s a_s} = \frac{757}{2 \times 78,5} = 4,8$$

Six stirrups are provided, three for each side of the secondary beam. The first stirrup is at the distance 50mm from the edge of the secondary beam, the distance between stirrups of the three stirrups is 50mm.

4.7. Calculation and construction of envelope of materials

a. Calculation of bending strength

At end spans, there are positive bending moments and the beam is in T-shaped cross section having flange within the compression zone. The effective width of the flange is $b = b'_f = 2650mm$. The longitudinal rebars provided at the bottom of the beam are $2\Phi 28 + 2\Phi 25$ with total cross-sectional area of $A_s = 2213mm^2$. The actual concrete cover to the center of these bars is $a = 44mm$, the effective depth is $h_0 = 700 - 44 = 656mm$.

$$\xi = \frac{R_s A_s}{R_b b h_0} = \frac{280 \times 2213}{8,5 \times 2650 \times 656} = 0,042$$

$$x = \xi h_0 = 0,042 \times 656 = 27,552mm < h'_f = 80mm - \text{Neutral axis is within the flange.}$$

$$\zeta = 1 - 0,5\xi = 1 - 0,5 \times 0,042 = 0,979$$

$$M_{td} = R_s A_s \zeta h_0 = 280 \times 2213 \times 0,979 \times 656 = 397,948 \times 10^6 Nmm = 397,948kNm.$$

At support B, there are negative bending moments and the beam is in rectangular $b \times h = 300 \times 700$ (mm). The longitudinal rebars provided at the top of the beam are $4\Phi 28$ (outer layer) and $2\Phi 22$ (interior layer) with total cross-sectional area of $A_s = 3223 \text{ mm}^2$. The distance from extreme concrete fiber under tension to centroid of the rebars group is $a = 67 \text{ mm}$, the effective depth is $h_0 = 700 - 67 = 633 \text{ mm}$.

$$\xi = \frac{R_s A_s}{R_b b h_0} = \frac{280 \times 3223}{8,5 \times 300 \times 633} = 0,559 < \xi_R = 0,65$$

$$\zeta = 1 - 0,5\xi = 1 - 0,5 \times 0,559 = 0,721$$

$$M_{td} = R_s A_s \zeta h_0 = 280 \times 3223 \times 0,7205 \times 633 = 411,582 \times 10^6 \text{ Nmm} = 411,582 \text{ kNm}.$$

Results of bearing capacity are presented in *Table 4.4*.

$$\xi = \frac{R_s A_s}{R_b b h_0}; \zeta = 1 - 0,5\xi; M_{td} = R_s A_s \zeta h_0$$

Table 4.4. Bearing capacity of sections

Section	Quantity of provided rebars (mm ²)	A _s (mm)	h ₀ (mm)	ξ	ζ	M _{td} (kNm)
Middle of end span	2Φ28+2Φ25	2213	656	0,042	0,979	397,948
Near to support of end span	Cut 2Φ28, remain 2Φ25	982	657	0,019	0,991	179,023
Over support B	2Φ28+2Φ28+2Φ22	3223	633	0,559	0,721	411,582
Beside support B	Cut 2Φ22, remain 2Φ28+2Φ28	2463	646	0,419	0,791	352,396
Beside support B	Cut 2Φ28, remains 2Φ28	1232	646	0,209	0,895	199,446
Middle of span 2	2Φ25+1Φ25	1473	657	0,028	0,986	267,179
Near to support of span 2	Cut 1Φ25, remain 2Φ25	982	657	0,019	0,991	179,023
Over support C	2Φ28+2Φ28	2463	646	0,419	0,791	352,396
Beside support C	Cut 2Φ28, remain 2Φ28	1232	646	0,209	0,895	199,446

b. Determination of theoretical cutoff section of longitudinal rebars

The cutoff point at the left and (near to support B) of rebars No.4 (1Φ25) should be determined. After the rebars No.4 are cut off, there are rebars No.3 (2Φ25) remaining on the bottom at the second span, the positive moment strength is 179,023kNm. The material resistance envelope intersects with the bending moment envelope at point H. This is the theoretical cutoff section of rebars No.4. The distance from point H to the right edge of the second support is determined based on the principle of similar triangles to be 1959mm (*Figure 4.8*).

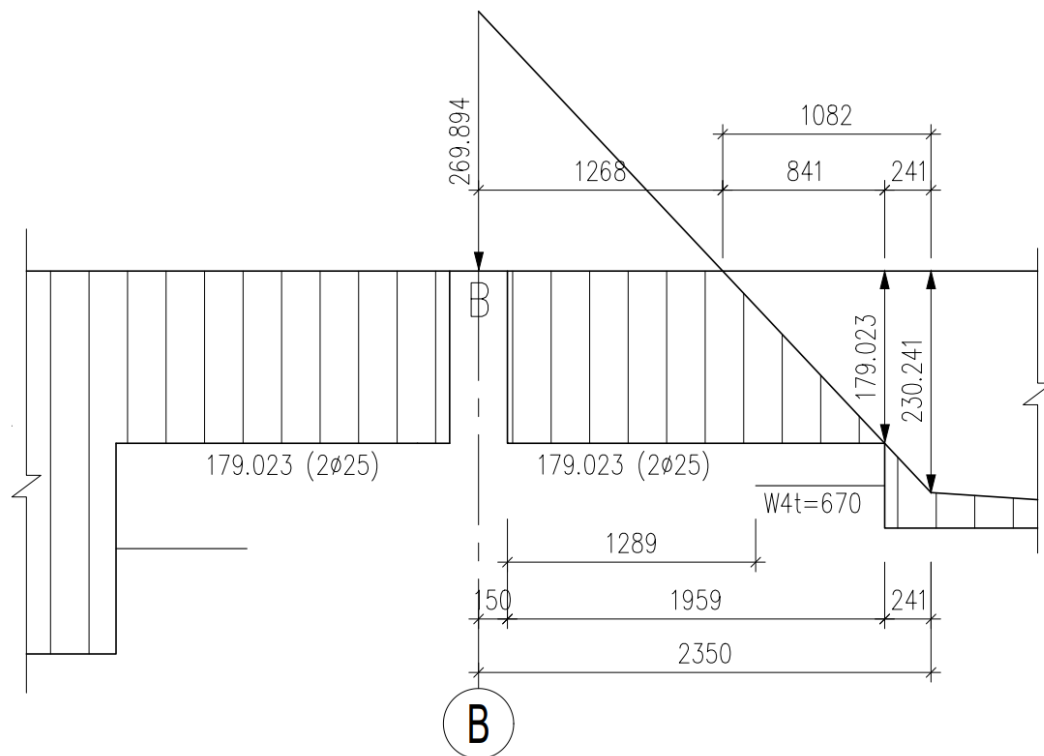


Figure 4.8. Theoretical cutoff section of rebar No.4

Determination of extension length W_4^t based on the principle of similar triangles and the corresponding shear force at point H, which is slope of moment diagram:

$$Q = \frac{269,894 + 230,241}{2.35} = 212,823 \text{ kN}$$

In this segment, stirrups provided are $\Phi 10$ a140 with $q_{sw} = 196,25 \text{ kN/m}$

There are no inclined rebars at this position, then $Q_{s.inc} = 0$

Hence:

$$W_4^t = \frac{Q - Q_{s.inc}}{2q_{sw}} + 5\varphi = \frac{212,823 - 0}{2 \times 196,25} + 5 \times 0,025 = 0,667m > 20\phi = 20 \times 0,025m = 0,5m$$

Choose $W_4^t = 670mm$

Similar calculations are conducted for cutoff points of the other rebars. The results are presented in Table 4.5.

Table 4.5. Theoretical cutoff section of longitudinal rebars

Rebars	Theoretical cutoff section	Extension length
No.2 (left end)	At a distance of 1088mm from the right of axis A	$W_2^t = 936mm$
No.2 (right end)	At a distance of 1883mm from the left of axis B	$W_2^p = 675mm$
No.4 (left end)	At a distance of 2109mm from the right of axis B	$W_4^t = 670mm$
No.4 (right end)	At a distance of 1892mm from the left of axis C	$W_4^p = 651mm$
No.5 (left end)	At a distance of 2672mm from the left of axis B	$W_5^t = 568mm$
No.6 (left end)	At a distance of 914mm from the left of axis B	$W_6^t = 723mm$
No.6 (right end)	At a distance of 1075mm from the right of axis B	$W_6^p = 568mm$
No.7 (left end)	At a distance of 342mm from the left of axis B	$W_7^t = 700mm$
No.7 (right end)	At a distance of 370mm from the right of axis B	$W_7^p = 738mm$
No.8 (left end)	At a distance of 731mm from the left of axis C	$W_8 = 789mm$

9. Anchorage of longitudinal rebars

The remaining bottom rebars anchored into the support shall be at least 1/3 in area compared to that of the longitudinal rebars at mid-span.

- End span:

$2\Phi 28 + 2\Phi 25$, cut $2\Phi 28$, remain $2\Phi 25$, anchored area of 44,36%.

- Interior span:

$2\Phi 25 + 1\Phi 25$, cut $1\Phi 25$, remain $2\Phi 25$, anchored area of 66,67%.

At support B:

$$Q_{max} = 267,014 < Q_{b,1} = 2,5R_{bt}bh_0 = 2,5 \times 0,75 \times 300 \times 633 = 356062,5kN$$

Calculate:

$$0,3L_{0,an} = 0,3\phi(R_s / R_{bt}) / (4\eta_1\eta_2) = 0,3 \times \phi \times (280 / 0,75) / (4 \times 2,5 \times 1) = 11,2\phi$$

$$\text{Hence: } l_{an} \geq \max(15\phi; 200; 0,3L_{0,an}) = \max(380; 200; 280) = 380mm$$

Anchorage length $l_{an} \geq 380mm$.

10. Distribution reinforcement

Rebars No.10 ($2\Phi 14$) having $A_s = 308mm^2 > 0,1\% \times 300 \times 633 = 190mm^2$. These contributed rebars are used for top side at end span, where there is no negative moment.

Rebars No.11 ($2\Phi 14$) having $A_s = 308mm^2 > 0,1\% \times 300 \times 633 = 190mm^2$. These contributed rebars are used at middle of the beam for shrinkage of concrete as well as to increase stability of the reinforcement cage when $h \geq 700mm$.

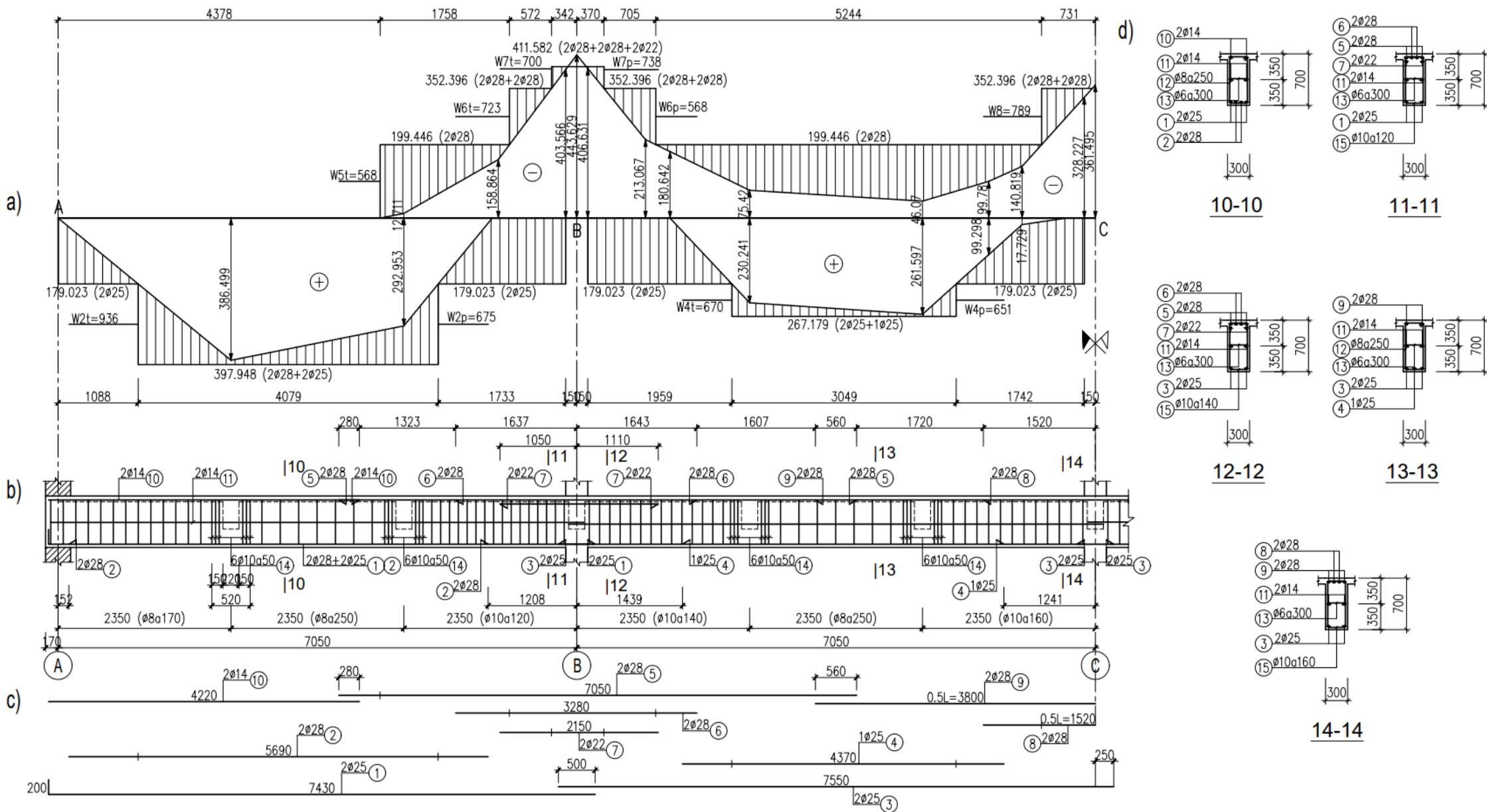


Figure 4.9. Reinforcement arrangement of primary beams

a) Material resistance envelope; b) Section; c) Rebar development; d) Cross sections