FACULTY OF CIVIL ENGINEERING, CTU IN PRAGUE

Commented example of combined precast and in-situ reinforced concrete slab design

Educational material

Ing. Josef Fládr

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1. Assignment

The goal of this commented example is to make the students of Faculty of Civil Engineering, CTU in Prague familiar with the design procedure of combined precast and in-situ slabs. Design of load-bearing members is shown also with respect to assembling stages of particular precast elements of the slab. The aim of the commented example is to show the complexity of structural design.

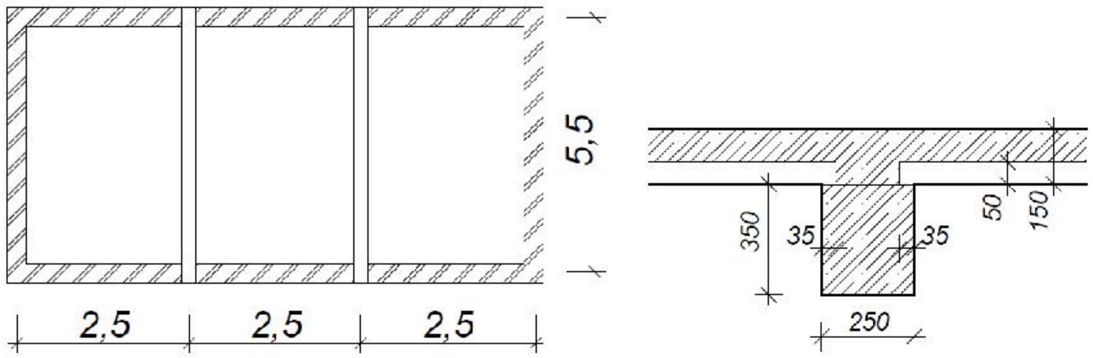


Fig. 1 Scheme of the structure

2. Design of the structure for service life

At the first step, the structure will be design for the loads that affect it during its service life, because these loads are the dominant ones.

2.1. Loading of the slab

Permanent	Thickness [mm]	Bulk density [kg/m^3]	Char. load [kN/m^2]	Partial factor	Design load [kN/m^2]
Ceramic tiles	8	2000	0,16	1,35	0,22
Flexible cement	2	2100	0,04	1,35	0,06
Concrete	100	2500	2,50	1,35	3,38
Self-weight	150	2500	3,75	1,35	5,06
Plaster	15	1800	0,27	1,35	0,36
Total of permanent load		Σ	**6,72**	Σ	**9,07**
Variable					
Category C4: Areas for physical activities			5,00	1,5	7,5
Removable partitions, weight < 3,0 kN/m			1,20	1,5	1,8
Total of variable		Σ	**6,20**	Σ	**9,3**
TOTAL			**12,92**		**18,4**

2.2. Calculation of cover depth

<u>Description of the structure</u>

- The structure is inside the building, with low-humidity operations => **XC1**
- The surface is not exposed to corrosion caused by chlorides
- The surface is not in contact with sea water
- The structure is protected, therefore is not exposed to freeze-thaw cycles
- There is no risk of chemical attack of the structure
- Slab-like structure

<u>Requirements:</u> XC1 …. C 30/37; w/c < 0,6, c > 260 kg/m^3
Serviceability life 50 years => structural class **S4**

$c_{nom} = c_{min} + \text{a}c_{dev}$
$c_{\min} = \max\{c_{\min,b};\ c_{\min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add};\ 10\text{ mm}\}$
$c_{\min,b}$ = 10 mm ***(estimated diameter of rebars)***
$c_{\min,dur}$ = 10 mm

Values of $c_{min,dur}$ [mm]							
Structural class	**Exposure class related to environmental conditions**						
	X0	**XC1**	**XC2/XC3**	**XC4**	**XD1/XS1**	**XD2/XS2**	**XD3/XS3**
S1	10	10	10	15	20	25	30
S2	10	**10**	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

$\Delta c_{dur,\gamma}$; $\Delta c_{dur,st}$; $\Delta c_{dur,add}$ considered equal to 0
$c_{\min,st}$ = max{10; 100 + 0 – 0 − 0; 10} = 10 mm
$\text{a}c_{dev}$ = 10 mm ***(precast structure)***
$c_{nom,st}$ = $c_{\min,st} + \text{a}c_{dev}$ = 10 + 10 = 20 mm

2.3. Material characteristics

CONCRETE C 30/37 =>

$$f_{ck} = 30\ MPa$$

$$f_{cd} = \frac{30}{1{,}5} = 20{,}0\ MPa$$

$$f_{ctm} = 2{,}9\ MPa$$

$$f_{ctk0,05} = 2{,}0\ MPa$$

$$£_{cu3} = 3{,}5‰$$

STEEL B500B =>

$$f_{yk} = 500\ MPa$$

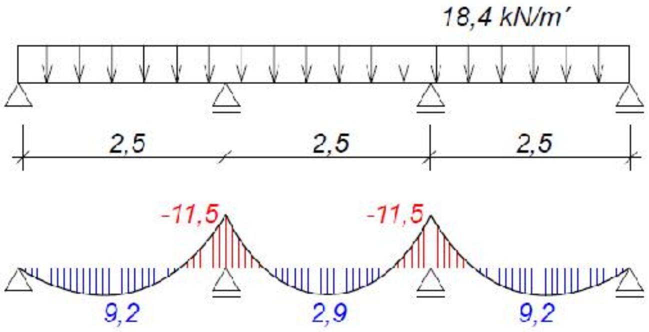
$$f_{yd} = \frac{500}{1{,}15} = 434\ MPa$$

$$E_s = 200\,000 MPa$$

$$£_s = \frac{f_{yd}}{E_s} = \frac{434}{200\ 000} = 2{,}174‰$$

$$¶_{bal} = \frac{£_{cu3}}{£_{cu3} + £_s} = \frac{3{,}5}{3{,}5 + 2{,}174} = 0{,}616$$

Moment curve



In final stage, filigree panels act as continuous beam. Therefore i tis necessary to place upper reinforcement above the supports (beams). This reinforcement will be placed into the slab dutiny casting of in-situ part of the structure.

2.4. ULS design of reinforcement – final design load

- Design of midspan reinforcement ***(positive moment)***

Estimated diameter of rebars **Ø 8**

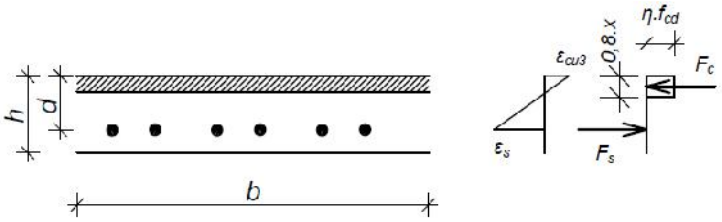


Fig. 2 Distribution of internal forces in slab (positive moment)

$d = h - c - Ø/2 = 150 - 20 – 8/2 = 126$ mm

$$\mu = \frac{m_{Ed}}{b.d_1^{\,2}.\eta.f_{cd}} = \frac{9{,}2}{1{,}0.0{,}126^2.1{,}0.20{,}0.10^3} = 0{,}0289 \Rightarrow \quad \zeta = 0{,}980 \quad \xi = 0{,}038$$

$$\xi = 0{,}038 \leq 0{,}45 = \xi_{\lim} \quad \Rightarrow \quad \sigma_s = f_{yd}$$

$$A_{s,req} = \frac{m_{Ed}}{f_{yd}.\zeta.d} = \frac{9{,}2 \cdot 10^6}{434{,}7.10^3.0{,}98.0{,}126} = 171{,}4\ mm^2$$

Design: Ø8 per 150 mm ($A_{s,prov}$ = 335 mm^2)

Check of spacing ***(using detailing rules)***

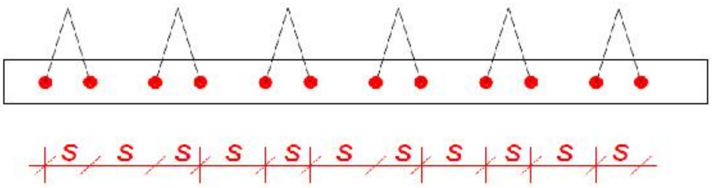


Fig. 3 Example of arrangement of lower reinforcement

$$s_{prov} = 150\ mm$$
$$s_{\max,1} = 2.h = 2.150 = 300\ mm \qquad s_{\max} = \min(s_{\max,1}; s_{\max,2}) = 300\ mm$$
$$s_{\max,2} = 300\ mm$$

$$s_{\min} = \max\ (1{,}2.Ø;\ d_g + 5;\ 20mm) = \max\ (1{,}2.8;\ 22 + 5;\ 20) \Rightarrow s_{\min} = 27mm$$

(d_g maximum diameter of aggregate grains)

$s_{\min} \le s_{prov} \le s_{\max}$

$27\ mm \le 150\ mm \le 300 mm$ ***CHECKED***

Check of minimum cross-sectional area of reinforcement

$A_{s,prov} = 335\ mm^2$

$A_{s,\max} = 0{,}04.A_c = 0{,}04.150.1000 = 6\ 000\ mm^2$

$$A_{s,\min} = \frac{0{,}26.f_{ctm.}b_t.d}{f_{yk}} = \frac{0{,}26.2{,}9.1000.126}{500} = 190{,}0\ mm^2$$

$A_{s,\min} \ge 0{,}0013.b_t.d = 0{,}0013.1000.126 = 163{,}8\ mm^2$

$A_{s,\min} \le A_{s,prov} \le A_{s,\max}$

$190{,}0 \le 335 \le 6\ 000$ ***CHECKED***

Check of reinforcement for ultimate limit state (ULS)

$$x = \frac{A_{s,prov} \cdot f_{yd}}{\lambda.b.\eta.f_{cd}} = \frac{335.434{,}7}{0{,}8.1000.1{,}0.20{,}0} = 9{,}1\ mm$$

$$\xi = \frac{x}{d} = \frac{9{,}1}{126} = 0{,}072 \le 0{,}45 = \xi_{\lim}$$

$z = d - 0{,}5.\lambda.x = 0{,}126 - 0{,}5.0{,}8.0{,}091 = 0{,}121$ m

$m_{Rd} = a_{s,prov} \cdot f_{yd}.z = 335.434{,}7.10^{-3}.0{,}121 = 17{,}69\ \ kNm$

$m_{Rd} = 17{,}69\ kNm \ge 9{,}2\ kNm = m_{Ed}$ ***CHECKED***

reserve 92%

- Reinforcement in support ***(negative moment)***

We choose Ø 10 ***diameter of upper reinforcement is usually one or two levels higher than for lower reinforcement***

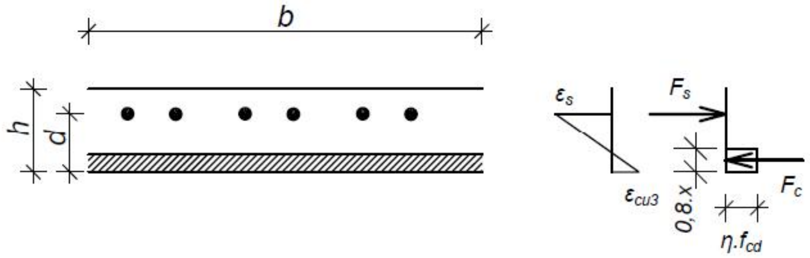


Fig. 4 Distribution of internal forces in slab (negative moment)

$d = h$ - c- $Ø /2$ = 150 - 20 – 10/2 = 125 mm

$$\mu = \frac{m_{Ed}}{b.d_1^{\,2}.\eta.f_{cd}} = \frac{11{,}48}{1{,}0.0{,}125^2.1{,}0.20{,}0.10^3} = 0{,}037 \Rightarrow$$

$\zeta = 0{,}982$

$\xi = 0{,}045$

$\xi = 0{,}045 \le 0{,}45 = \xi_{\lim} \ \Rightarrow\ \sigma_s = f_{yd}$

$$A_{s,req} = \frac{m_{Ed}}{f_{yd}\,\zeta.d} = \frac{11{,}48 \cdot 10^6}{434{,}7.10^3.0{,}982.0{,}125} = 215{,}2\ mm^2$$

Design: Ø10 per 300 mm ($A_{s,prov}$ = 262 mm^2)

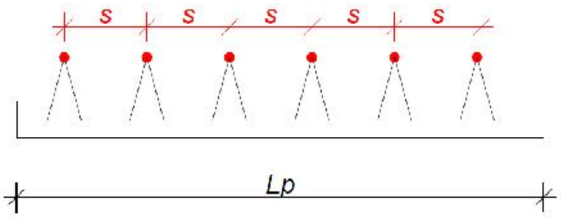


Fig. 5 Example of arrangement of upper reinforcement

Check of spacing

$s_{prov} = 300\ mm$

$s_{\max,1} = 2.h = 2.150 = 300\ mm$

$s_{\max,2} = 300\ mm$

$s_{\max} = \min(s_{\max,1}; s_{\max,2}) = 300\ mm$

$s_{\min} = \max\ (1{,}2.¤;\ d_g + 5;\ 20mm) = \max\ (1{,}2.10;\ 22+5;\ 20) \Rightarrow s_{\min} = 27mm$

(d_g maximum diameter of aggregate grains)

$s_{\min} \le s_{prov} \le s_{\max}$

$27\ mm \le 300\ mm \le 300mm$ **CHECKED**

Check of minimum cross-sectional area of reinforcement

$A_{s,prov} = 262\ mm^2$

$A_{s,\max} = 0{,}04.A_c = 0{,}04.150.1000 = 6\ 000\ mm^2$

$A_{s,\min} = \frac{0{,}26.f_{ctm}.b_t.d}{f_{yk}} = \frac{0{,}26.2{,}9.1000.125}{500} = 188\ mm^2$ *(brittle failure)*

$A_{s,\min} \ge 0{,}0013.b_t.d = 0{,}0013.1000.125 = 163\ mm^2$ *(reinforcement ratio)*

$A_{s,\min} \le A_{s,prov} \le A_{s,\max}$

$188 \le 262 \le 6\ 000$ **CHECKED**

Check of reinforcement in ultimate limit state (ULS)

$x = \frac{A_{s,prov} \cdot f_{yd}}{ª.b.¦ \cdot f_{cd}} = \frac{262.434{,}7}{0{,}8.1000.1{,}0.20{,}0} = 7{,}1\ mm$

$¶ = \frac{x}{d} = \frac{7{,}1}{125} = 0{,}056 \le 0{,}45 = ¶_{\lim}$

$z = d - 0{,}5.\lambda.x = 0{,}125 - 0{,}5.0{,}8.0{,}071 = 0{,}121$ m

$m_{Rd} = a_{s,prov} \cdot f_{yd}.z = 262.434{,}7.10^{-3}.0{,}121 = 13{,}83\ \ kNm$

$m_{Rd} = 13{,}83\ kNm \ge 11{,}48\ kNm = m_{Ed}$ **CHECKED**

reserve 20%

2.5. SLS check of panel

For the purpose of this example, serviceability limit state will be checked using simplified method based on deflection control criterion.

Concrete class: **C 30/37**
Type of structure: **continuous beam** ⟶ $\lambda_{d,tab}$ = 30,8 pro ρ = 0,5%
$\lambda_{d,tab}$ = 21,0 pro ρ = 1,5%
$\boldsymbol{A_{s,prov}}$ **= 335 mm^2** *(lower reinforcement is crucial for deflections)*
$\boldsymbol{A_{s,req}}$ **= 171 mm^2** *(required cross-sectional area of reinforcement from ULS)*
Span: ***l* = 2,5 m**
Reinforcement ratio $\rho = A_{s,prov} / A_c$ = 335 / (1000.150) = 0,22 %

As the reinforcement ratio of our panel is less than 0,5 %, $\lambda_{d,tab}$ can be taken as 30,8 and no interpolation is required.

$\lambda_{d,tab}$ = 30,8 pro ρ = *0,5%*
κ_{c1} = 1,0 *(rectangular cross-section)*
κ_{c2} = 1,0 *(span is less than 7,0 m)*

$$\kappa_{c2} = \frac{500}{f_{yk}} \cdot \frac{A_{s,prov}}{A_{s,req}} = \frac{500}{500} \cdot \frac{335}{171} = 1{,}96$$

$\lambda_d = \kappa_{c1} . \kappa_{c2} . \kappa_{c3} . \lambda_{d,tab}$ = 1,0 . 1,0 . 1,96 . 21,00 = **41,16 –**
l / d = 2,5 / 0,126 = **19,84 –**
$l / d < \lambda_d$
19,84 < 41,16 => Checked

3. Check of the beam

3.1. Calculation of cover depth

Description of the structure
- The structure is inside the building, with low-humidity operations => **XC1**
- The surface is not exposed to corrosion caused by chlorides
- The surface is not in contact with sea water
- The structure is protected, therefore is not exposed to freeze-thaw cycles
- There is no risk of chemical attack of the structure

Requirements: XC1 …. C 30/37; w/c < 0,6, c > 260 kg/m^3
Serviceability life 50 years => structural class **S4**

- Cover depth for stirrups

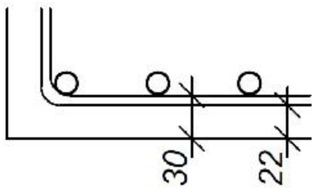
$c_{nom} = c_{min} + ac_{dev}$
$c_{min} = \max \{c_{min,b}; c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10\text{ mm}\}$
$c_{min,b}$ = 8 mm *(estimated diameter of stirrups)*
$c_{min,dur}$ = 10 mm

Values of $c_{min,dur}$ [mm]							
Structural class	**Exposure class related to environmental conditions**						
	X0	**XC1**	**XC2/XC3**	**XC4**	**XD1/XS1**	**XD2/XS2**	**XD3/XS3**
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	**10**	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

$\Delta c_{dur,\gamma}$; $\Delta c_{dur,st}$; $\Delta c_{dur,add}$ considered equal to 0
$c_{min,st}$ $= \max\{8; 10 + 0 - 0 - 0; 10\} = 20$ mm
ac_{dev} $= 10$ mm ***(precast structure)***
$c_{nom,st}$ $= c_{min,st} + ac_{dev} = 10 + 10 = 20$ mm

- Cover depth for main bending reinforcement

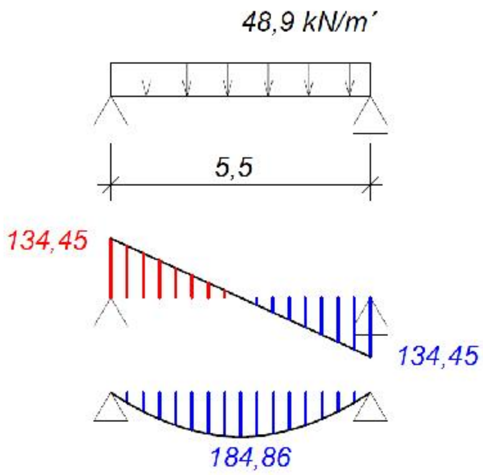
$c_{nom} = c_{min} + ac_{dev}$
$c_{min} = \max\{c_{min,b}; c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10 \text{ mm}\}$
$c_{min,b}$ $= 22$ mm ***(estimated diameter of rebars)***
$c_{min,dur}$ $= 10$ mm
$\Delta c_{dur,\gamma}$; $\Delta c_{dur,st}$; $\Delta c_{dur,add}$ considered equal to 0
$c_{min,st}$ $= \max\{22; 10 + 0 - 0 - 0; 10\} = 22$ mm
ac_{dev} $= 5$ mm ***(precast structure)***
$c_{nom,st}$ $= c_{min,st} + ac_{dev} = 22 + 5 = 27 \approx 30$ mm



3.2. Design of main bending reinforcement in midspan

- **Calculation of loads**

(18,4.2,5+0,25.0,35.25.1,35) = 48,9 kN/m´

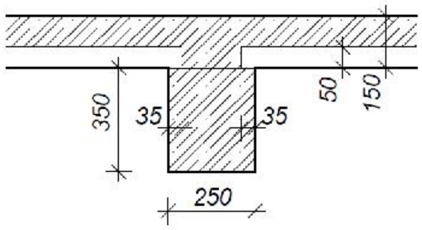


Slab is supported by masonry walls, therefore it acts as a simple beam.

The beam is loaded by the load from the slab multiplied by loading width of the beam (axial span in our case) + the self weight of part of the beam under the slab.

$$M_{Ed,midspan} = 184{,}86\ kNm$$

We choose Ø22



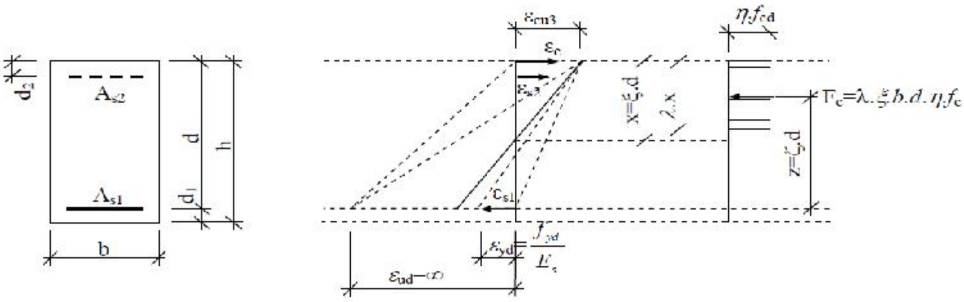


Fig. 6 Distribution of internal forces in beam (positive moment)

$d = h - c - \frac{Ř}{2} = 500 - 30 - \frac{22}{2} = 459\ mm$ *(In final design state, beam collaborates with slab)*

$« = \frac{M_{Ed}}{b.d^{2}.f_{cd}} = \frac{184{,}86}{0{,}25.0{,}459^{2}.20{,}0.10^{3}} = 0{,}175 \Rightarrow$ $\xi = 0{,}245$

$\zeta = 0{,}902$

$\xi = 0{,}245 \le 0{,}45 = \xi_{lim}$ ***CHECKED***

$A_{s,req} = \frac{M_{Ed}}{f_{yd}.s.d} = \frac{184{,}86.10^{3}}{434.10^{6}.0{,}902.0{,}459} = 1028 mm^{2}$

Check of reinforcement ratio

$A_{s,prov} = 1140\ mm^{2}$

$A_{s,\max} = 0{,}04.A_{c} = 0{,}04.500.250 = 5000\ mm^{2}$

$A_{s,\min} = \frac{0{,}26.f_{ctm}.b_{w}.d}{f_{yk}} = \frac{0{,}26.2{,}9.250.459}{500} = 173\ mm^{2}$

$A_{s,\min} \ge 0{,}0013.b_{w}.d = 0{,}0013.250.459 = 149\ mm^{2}$

$A_{s,\min} \le A_{s,prov} \le A_{s,\max}$

$173 \le 1140 \le 5000$ ***CHECKED***

Check of spacing

$s_{prov} = \frac{b_{w} - 2.c - n.¤}{n-1} = \frac{250 - 2.30 - 3.22}{2} = 62\ mm$

$s_{\min} = \max\ (1{,}2.¤; d_{g} + 5; 20mm) = \max\ (1{,}2.22; 16 + 5; 20) \Rightarrow s_{\min} = 26{,}4mm$

(d_g maximum diameter of aggregate grains)

$s_{\min} \le s_{prov} \le s_{\max}$

$26{,}4\ mm \le 62\ mm$ ***CHECKED***

Check of reinforcement in ultimate limit state (ULS)

Design: 3Ø22 ($A_{s,prov}$ = 1140 mm²)

$$x = \frac{A_{sd} \cdot f_{yd}}{0{,}8.b.f_{cd}} = \frac{1140.10^{-6}.434}{0{,}8.0{,}25.20{,}0} = 124\ mm$$

$$\P = \frac{x}{d} = \frac{0{,}124}{0{,}459} = 0{,}270 \leq 0{,}45 = \P_{\lim}$$ **CHECKED**

$z = d - 0{,}4.x = 459 - 0{,}4.\ 124 = 409$ mm

$$M_{Rd} = A_s \cdot f_{yd}.z = 1140.10^{-6}.434.0{,}409 = 194{,}32\ kNm$$

$$M_{Rd} = 194{,}32\ kNm \geq 184{,}86\ kNm = M_{Ed}$$ **CHECKED**

reserve 5%

3.3. Design of shear reinforcement

V_{Ed} = 134,4 kNm *(maximum value of shear force)*

Capacity of compressed concrete strut

$\cot\bar{} = 1{,}5$ *(recommended value for optimum slope of shear cracks)*

$$v = 0{,}6\left(1 - \frac{f_{ck}}{250}\right) = 0{,}6.\left(1 - \frac{30}{250}\right) = 0{,}528$$

$$V_{Rd,\max} = v.f_{cd}.b_w.z\frac{\cot\bar{}}{1+\cot^2\bar{}} = 0{,}528.20{,}0.0{,}25.0{,}409.\frac{1{,}5}{1+1{,}5^2} = 356{,}9\ kN$$

(maximum load-bearing capacity of concrete in compression perpendicular to anticipated shear crack)

Design of stirrups

Ø	**8**
number of legs	**2**
s	**150 mm**
$f_{y,wk}$	**500 MPa**

$$^\circ{}_{w,\min} = 0{,}08.\frac{\sqrt{f_{ck}}}{f_{y,wk}} = 0{,}08.\frac{\sqrt{30}}{500} = 0{,}00088$$

$$s_{1,\max} = \frac{A_{sw}}{b_w.{}^\circ{}_{w,\min}} = \frac{2.50{,}6}{250.0{,}00088} = 460\ mm$$

$$s_{2,\max} = \min(0{,}75.d;400) = 0{,}75.459 = 344\ mm$$

$$s_{\max} = \min\ (460;\ 344) = 344\ mm$$ *(maximum spacing of stirrups according to detailing rules)*

$$s_{prov} = 150\ mm \leq 344\ mm = s_{\max}$$ **CHECKED**

$$V_{Rd,s} = \frac{n.A_{sw} \cdot f_{ywd}}{s}.z.\cot g^{-} = \frac{4.50,26.500/1,15}{150}.459.1,5 = 171,4\ kN$$

$$V_{Ed} = 134,4\ kN \leq 171,4\ kN = V_{Rd} \leq 356,9\ kN = V_{Rd,\max}$$

CHECKED
reserve 27%

3.4. Connection

In case of full connection between beam and slab, the strain in critical cross-section is continuous up to the stage when limit elastic state of the cross-section is reached. This situation can be best characterized as a stage when yield limit of tensile reinforcement is reached (the strain of reinforcement is $\boldsymbol{\varepsilon_{sy}}$). Load-bearing capacity of such cross-section is given by force couple $\boldsymbol{N_c}$ and $\boldsymbol{N_{sy}}$ with lever arm $\boldsymbol{z}$.

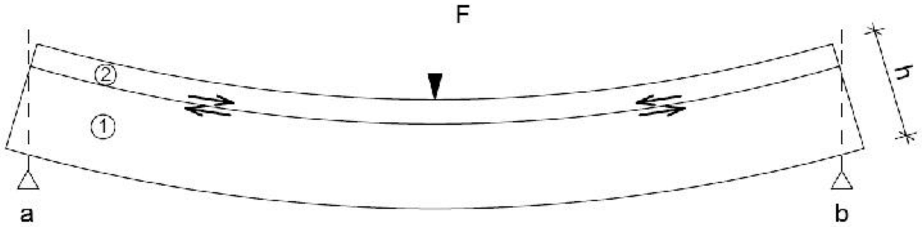


Fig. 6 Behavior of two fully connected members subjected to bending

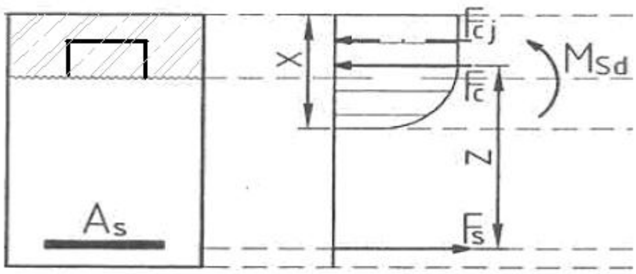
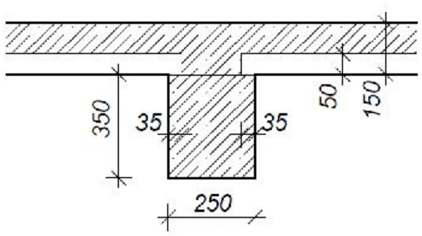


Fig. 7 Distribution of internal forces in precast and in-situ part of the cross-section.

$b_i = b_w - 2.u_p = 250 - 2.35 = 180$mm ***(effective width of the beam for transmission of shear stress between precast and in-situ part of the structure)***



Minimum bedding length of filigree panel on the beam is 35 mm. From technological point of view, even smaller values are possible, but temporary linear support right next to the beam is required in such a case. This option requires a way of support different from the one shown in this example.

Reduction of shear forces (for optimization of results)

$$V_{Ed,1} = V_{Ed} - \left(\left(\frac{b}{2} + d \right) \cdot (g+q)_d \right) = 134,45 - \left(\left(\frac{0,250}{2} + 0,459 \right) \cdot (48,9) \right) = 105,9\ kN$$

$$v_{Ed} = \frac{\cdot V_{Ed,1}}{z \cdot b_i} = \frac{1{,}0 \cdot 105{,}9 \cdot 10^3}{409 \cdot (250 - 2 \cdot 35)} = 1{,}43\ MPa$$

is ratio between longitudinal force in in-situ part of cross-section and total longitudinal force in compressed part of cross-section, in our case = 1,0, because neutral axis lays in in-situ part of cross-section (see check of beam),

z is lever arm of internal forces (see check of beam),

b_i is width of connection area between in-situ and precast concrete members,

$$v_{Rd} = c \cdot f_{ctd} + \text{«} \cdot \pm_n + \text{°} \cdot f_{yd} \cdot (\text{«} \cdot \sin\text{~} + \cos\text{~}) \leq 0{,}5 \cdot \neg \cdot f_{cd}$$

Connection type: rough connection

c = 0,45 ; μ = 0,7

where:

c, « are coefficients depending of roughness of connection area,

f_{ctd} is design tensile strength of concrete calculated from characteristic strength $f_{ctk,0.05}$

$\pm_n$ is normal stress perpendicular to connection area, in our case we take $\pm_n = 0$,

°: is reinforcement ratio of connection area by connecting members,

α is angle between connecting reinforcement and shear plane, in our case reinforcement will be perpendicular to concrete surface and therefore α = 90°,

ν is reduction factor for compressive strength of concrete in shear, $\neg = 0{,}6\left(1 - \frac{f_{ck}}{250}\right)$

$$\neg = 0{,}6 \cdot \left(1 - \frac{30}{250}\right) = 0{,}528$$

$$v_{Rd} = 0{,}45 \cdot \frac{2{,}0}{1{,}5} + 0{,}7 \cdot 0 + \frac{100{,}5.6{,}66}{250 \cdot 1000} \cdot 434{,}7 \cdot (0{,}7 \cdot 1 + 0) \leq 0{,}5 \cdot 0{,}528 \cdot 20$$

$v_{Rd} = 1{,}45\ MPa \leq 5{,}28\ MPa$ ***CHECKED***

$v_{Ed} = 1{,}43\ MPa \leq 1{,}45\ MPa = v_{Rd}$ ***CHECHED***

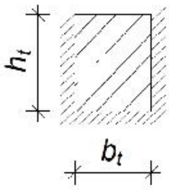
4. Temporary design situations

4.1. Design of lifting clutches of the beam

$F_P = V \cdot \text{°} = 0{,}25 \cdot 0{,}35 \cdot 5{,}5 \cdot 25 = 12{,}03\ kN$ *(self-weight of the beam)*

$F_{adh} = A \cdot q = [(0{,}35 + 0{,}25 + 0{,}35) \cdot 5{,}5 + (0{,}35 \cdot 0{,}25) \cdot 2] \cdot 1 = 5{,}4\ kN$ *(increase in self-weight due to adhesion to formwork)*

Adhesion coefficient for formwork	
Smooth, oiled formwork	q = 1 kN/m^2
Smooth, non-oiled formwork	q = 2 kN/m^2
Rough formwork	q = 3 kN/m^2



To minimize the labour consumption in production process, formwork on all sides of the panel is considered. This is also the worst case for calculation.

$N_{d,1} = \text{¢} . \dfrac{\text{¥}_{go}}{n.\cos\text{~}} . (F_p + F_{tah}) = 1{,}3 \cdot \dfrac{1{,}35}{2 \cdot \cos 30} \cdot (12{,}03 + 5{,}4) = 11{,}26\ kN$ *(force in one lifting clutch during formwork stripping)*

$N_{d,2} = \text{¢} . \dfrac{\text{¥}_{go}}{n.\cos\text{~}} . F_p = 2 \cdot \dfrac{1{,}35}{2 \cdot \cos 30} \cdot 12{,}03 = 13{,}13\ kN$ *(force in one lifting clutch during manipulation on construction site)*

$N_d = \max(N_{d,1}; N_{d,2}) = \max(11{,}26; 13{,}13) = 13{,}13\ kN$

During manipulation, the member is subjected to dynamic effects as well. These effects are taken into account by dynamic coefficient ***δ***, which increases static effects. The size of dynamic coefficient is given by lifting device and its rectification.

Recommended values of dynamic coefficient	
Fixed crane, rail crane < 90m/min	1,0 – 1,2
Fixed crane, rail crane > 90m/min	1,3 – 1,4
Lifting and transport in flat terrain	1,5 – 1,65
Lifting and transport in rough terrain (construction site)	> 2,0

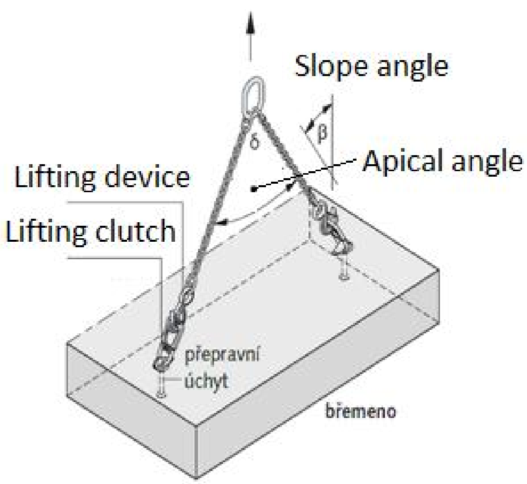
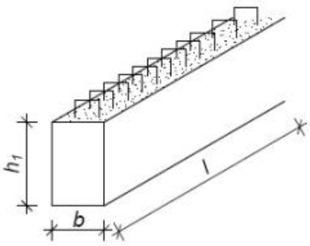


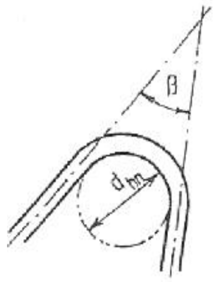
Fig. 8 Lifting of reinforced concrete panel

4.2. Number of connecting elements for manipulation



If the connecting reinforcement protrudes from the surface, they can be used for manipulation and no further lifting clutches are required.

Force in one leg of the connecting element



$$N = 0{,}6 \cdot \frac{N_d}{\cos(\quad +30)} = 0{,}6 \cdot \frac{13{,}13}{\cos(45+30)} = 21{,}56\ kN$$

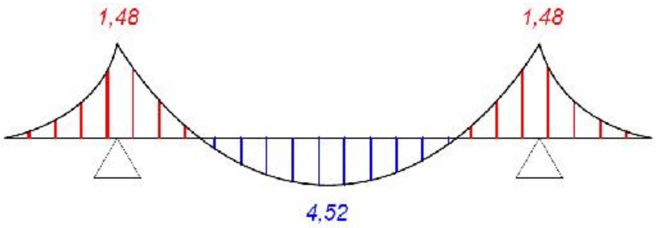
$$\text{¡} = 0{,}05 \cdot \text{¤} + 0{,}3 = 0{,}05 \cdot 8 + 0{,}3 = 0{,}7$$

$$A_{s,req} = \frac{N_d}{\text{¡} \cdot f_{yd}} = \frac{21{,}56}{0{,}7 \cdot 434{,}7} = 70{,}8\ mm^2$$

Cross-sectional area of one leg of stirrup is Ø8=50,3 mm^2

Required number of stirrups n = 70,8/50,3=1,4 => 2 kusy

4.3. Check of beam during transport



For manipulation, lifting clutches placed 1 m from the end of the beam will be used.

$$M_{Ed,v\,poli} = 4{,}52\ kNm$$

$$d = h - c - \frac{\text{Ř}}{2} = 350 - 30 - \frac{22}{2} = 309\ mm$$ *(only precast part is effective during transport)*

- Check of reinforcement

Návrh 3Ø22 ($A_{s,prov}$ = 1140 mm^2) *(already designed)*

$$x = \frac{A_{sd} \cdot f_{yd}}{0{,}8.b.f_{cd}} = \frac{1140.10^{-6}.434}{0{,}8.0{,}25.20{,}0} = 124\ mm$$

$$\text{¶} = \frac{x}{d} = \frac{0{,}124}{0{,}309} = 0{,}40 \le 0{,}617 = \text{¶}_{\max}$$ ***CHECKED***

$z = d - 0{,}4.x = 309 - 0{,}4.\ 124 = 256$ mm

$M_{Rd} = A_s \cdot f_{yd}.z = 1140.10^{-6}.434.0{,}256 = 128{,}34\ kNm$

$M_{Rd} = 128{,}34\ kNm \geq 4{,}52\ kNm = M_{Ed}$ **CHECKED**

$M_{Ed,negative} = 1{,}48\ kNm$

- Design of support reinforcement

We choose 2Ø10 ***(minimum beam reinforcement required by detailing rules)***

$d = h - c - \frac{Ř}{2} = 350 - 30 - \frac{10}{2} = 315\ mm$ ***(only precast part is effective during transport)***

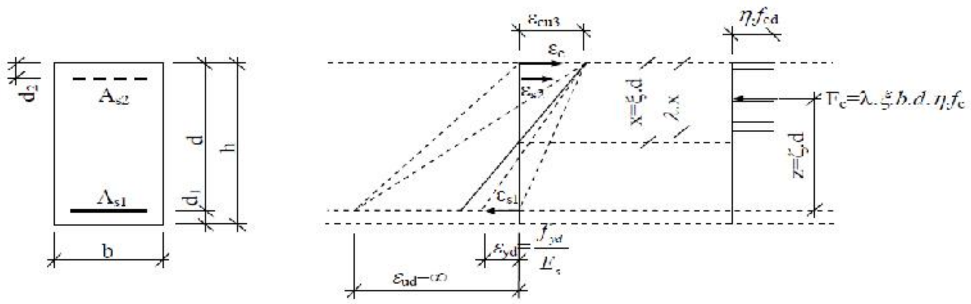


Fig. 9 Distribution of internal forces in beam (positive moment)

$« = \frac{M_{Ed}}{b.d^2.f_{cd}} = \frac{1{,}48}{0{,}25.0{,}315^2.20{,}0.10^3} = 0{,}003 \Rightarrow$ $\xi = 0{,}013$
$\zeta = 0{,}995$

$\xi = 0{,}013 \leq 0{,}45 = \xi_{lim}$ **CHECKED**

$A_{s,req} = \frac{M_{Ed}}{f_{yd}.ѕ.d} = \frac{1{,}48.10^3}{434.10^6.0{,}995.0{,}315} = 10{,}8\ mm^2$

- Check of reinforcement

Design: 2Ø10 ($A_{s,prov}$ = 157 mm^2)

$x = \frac{A_{sd} \cdot f_{yd}}{0{,}8.b.f_{cd}} = \frac{157.10^{-6}.434}{0{,}8.0{,}25.20{,}0} = 17\ mm$

$¶ = \frac{x}{d} = \frac{0{,}017}{0{,}315} = 0{,}05 \leq 0{,}617 = ¶_{\max}$ **CHECKED**

$z = d - 0{,}4.x = 315 - 0{,}4.\ 17 = 308$ mm

$M_{Rd} = A_s \cdot f_{yd}.z = 157.10^{-6}.434.0{,}308 = 20{,}98\ kNm$

$M_{Rd} = 20{,}98\ kNm \geq 1{,}48\ kNm = M_{Ed}$ **CHECKED**

4.4. Design of lifting clutches of filigree panel

$F_P = V \cdot ° = 0{,}05 \cdot 1{,}2 \cdot 2{,}5 \cdot 25 = 3{,}75\ kN$ ***(self-weight of the panel)***

$F_{adh} = A \cdot q = 1{,}2 \cdot 2{,}5 \cdot 1 = 3{,}25\ kN$ ***(increase in self-weight due to adhesion to formwork)***

$$N_{d,1} = \text{¢}.\frac{\text{¥}_{go}}{n.\cos\sim}.\left(F_p + F_{tah}\right) = 1{,}3 \cdot \frac{1{,}35}{2 \cdot \cos 30} \cdot \left(3{,}75 + 3{,}25\right) = 6{,}55\ kN$$ *(force in one lifting clutch during formwork stripping)*

$$N_{d,2} = \text{¢}.\frac{\text{¥}_{go}}{n.\cos\sim}.F_p = 2 \cdot \frac{1{,}35}{2 \cdot \cos 30} \cdot 3{,}25 = 4{,}09\ kN$$ *(force in one lifting clutch during manipulation on construction site)*

$$N_d = \max\left(N_{d,1}; N_{d,2}\right) = \max\left(6{,}55; 4{,}09\right) = 6{,}55\ kN$$

- Required number of stirrups

$$N = 0{,}6 \cdot \frac{N_d}{\cos(\quad + 30)} = 0{,}6 \cdot \frac{6{,}55}{\cos(45 + 30)} = 9{,}25\ kN$$

$$\text{¡} = 0{,}05 \cdot \text{¤} + 0{,}3 = 0{,}05 \cdot 8 + 0{,}3 = 0{,}7$$

$$A_{s,req} = \frac{N_d}{\text{¡} \cdot f_{yd}} = \frac{9{,}25}{0{,}7 \cdot 434{,}7} = 35{,}5\ mm^2$$

Cross-sectional area of one leg of stirrup Ø6=28,3 mm^2

Required number of stirrups n = 35,5/28,3=1,2 => 2 kusy

5. Temporary supports during concreting

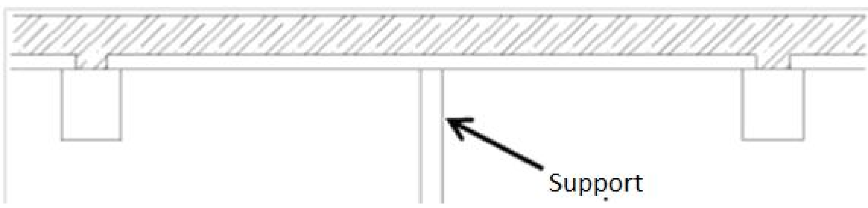
5.1. Filigree panel supports

Assembling loading

Permanent	Thickness [mm]	Bulk density [kg/m^3]	Char. load [kN/m^2]	Partial factor	Design load [kN/m^2]
In-situ concrete layer	100	2500	2,5	1,35	3,38
Filigree panel	50	2500	1,25	1,35	1,69
Total of permanent load		∑	3,75	∑	5,06
Variable					
Variable assembling load			0,75	1,5	1,13
Total of variable load		∑	0,75	∑	1,13
TOTAL		∑	**4,05**	∑	**6,19**

The standard determines variable load 0,75 kN/m^2 during assembling

The calculation is based on an assumption of one temporary support in the middle of span of filigree panel.



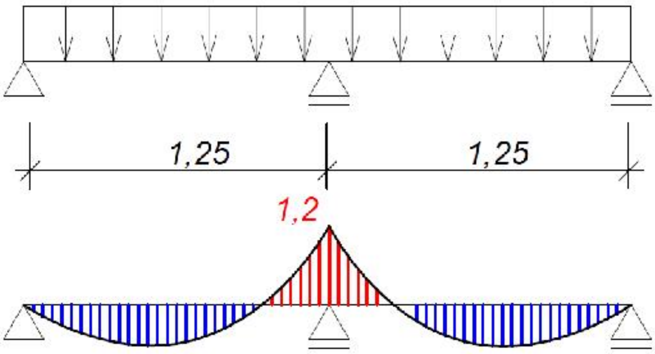


Fig. 10 Moment curve with temporary support

$$M_{Ed} = \frac{1}{8}\cdot(g+q)_d\cdot\left(\frac{l}{2}\right)^2 = \frac{1}{8}\cdot 6{,}19\cdot\left(\frac{2{,}5}{2}\right)^2 = 1{,}20\ kNm$$

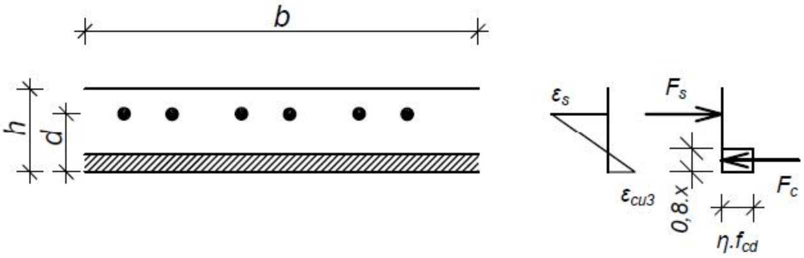


Fig. 11 Distribution of internal forces in slab (negative moment)

- Check of reinforcement in ULS

Ø10 per 300 mm ($A_{s,prov}$ = 262 mm^2) *(already designed)*

d = *h* - *c*- *Ø* /2 = 50 - 20 – 8/2 = 26 mm

$$x = \frac{A_{s,prov}\cdot f_{yd}}{\text{ª}.b.\text{¦}\cdot f_{cd}} = \frac{262.434{,}7}{0{,}8.1000.1{,}0.20{,}0} = 7{,}1\ mm$$

$$\text{¶} = \frac{x}{d} = \frac{7{,}1}{26} = 0{,}27 \le 0{,}617 = \text{¶}_{\lim}$$

$z = d - 0{,}5.\lambda.x = 26 - 0{,}5.0{,}8.7{,}1 = 23{,}2$ mm

$$m_{Rd} = a_{s,prov}\cdot f_{yd}.z = 262.434{,}7.10^{-3}.0{,}023 = 2{,}6\ \ kNm$$

$$\underline{m_{Rd} = 2{,}6\ kNm \ge 1{,}20\ kNm = m_{Ed}}$$ ***CHECKED***

reserve 116%

One support in the midspan is required. *Assumption checked.*

5.2. Beam supports

Loading area s = 2,5/2 =1,25 m

$$M_{Ed} = \frac{1}{8}\cdot\left[(g+q)_d\cdot s\right]\cdot\left(\frac{l}{2}\right)^2 = \frac{1}{8}\cdot 6{,}19\cdot 1{,}25\cdot 5{,}5^2 = 29{,}25\ kNm$$

- Design of midspan reinforcement

$$d = h - c - \frac{Ř}{2} = 350 - 30 - \frac{22}{2} = 309 \ mm$$ ***(only precast part is effective during transport)***

- Check of reinforcement

Design: 3 Ø22 ($A_{s,prov}$ = 1140 mm^2) ***(already designed)***

$$x = \frac{A_{sd} \cdot f_{yd}}{0{,}8.b.f_{cd}} = \frac{1140.10^{-6}.434}{0{,}8.0{,}25.20{,}0} = 124 \ mm$$

$$¶ = \frac{x}{d} = \frac{0{,}124}{0{,}309} = 0{,}40 \leq 0{,}617 = ¶_{\max}$$ ***CHECKED***

$z = d - 0{,}4.x$ = 309 - 0,4. 124 = 256 mm

$$M_{Rd} = A_s \cdot f_{yd} . z = 1140.10^{-6}.434.0{,}256 = 128{,}34 \ kNm$$

$$M_{Rd} = 128{,}34 \ kNm \geq 29{,}25 \ kNm = M_{Ed}$$ ***CHECKED***

The beam does not require temporary supports.