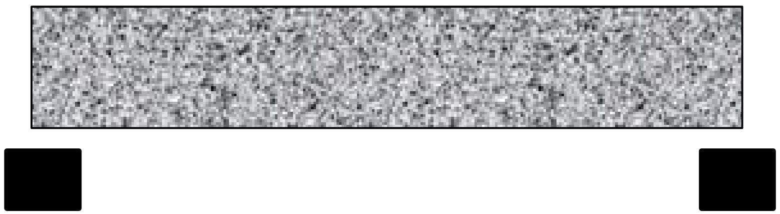
Curse Notes



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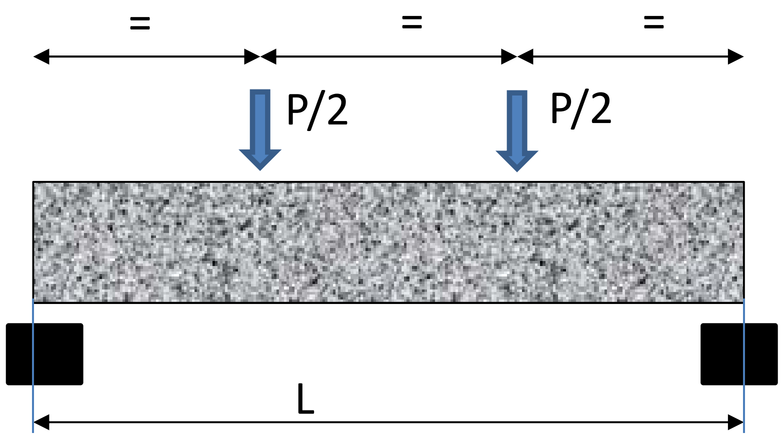
SIMPLE SUPORTED BEAM

b = 15 cm
h = 30 cm
L = 3,30 m
l_{rez}= 30 cm
P = 7 tones

Concrete	**C20/25**
Steel	**S500**
Exposure class	**XC1**



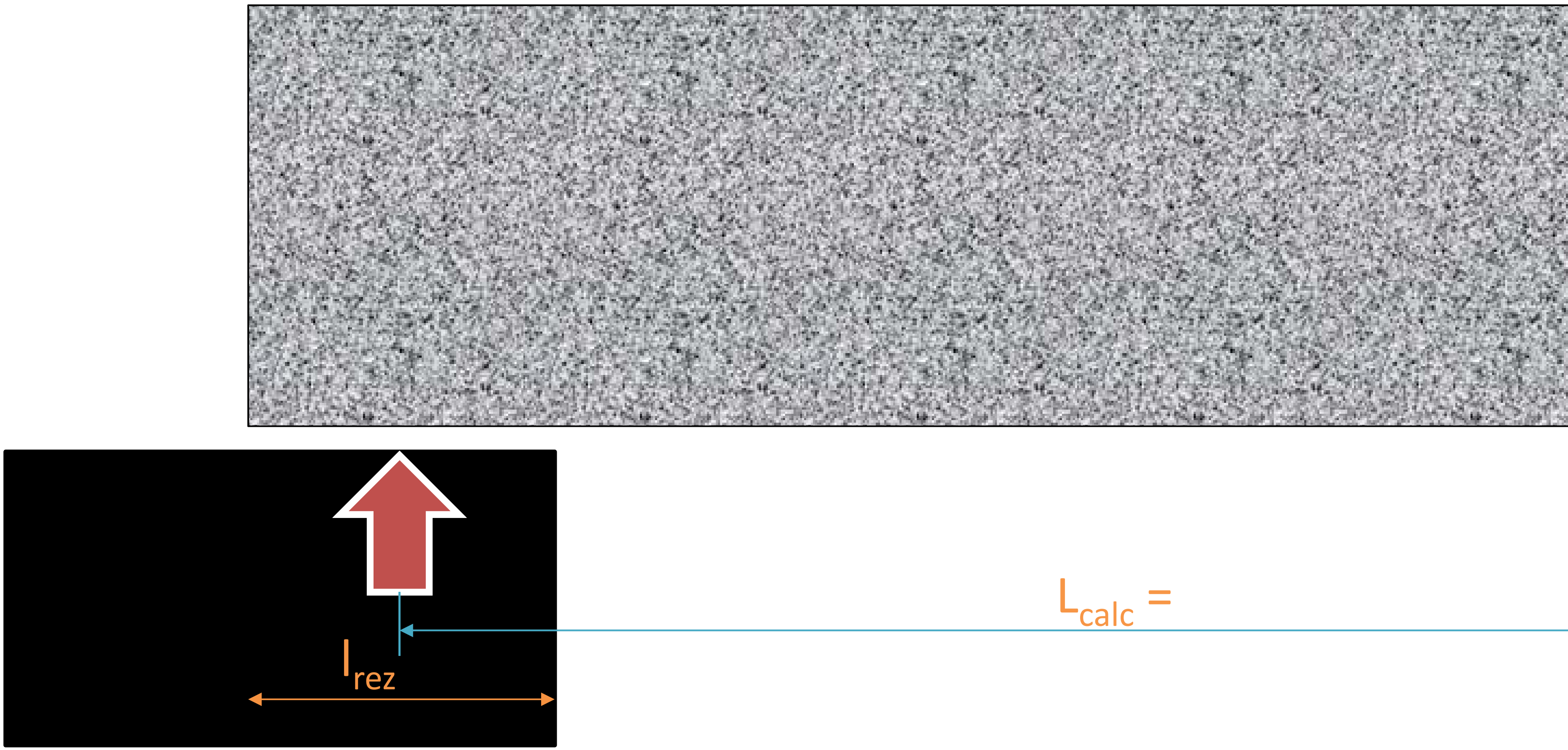
A_s = ?

A_{sw} = ?

s_w = ?

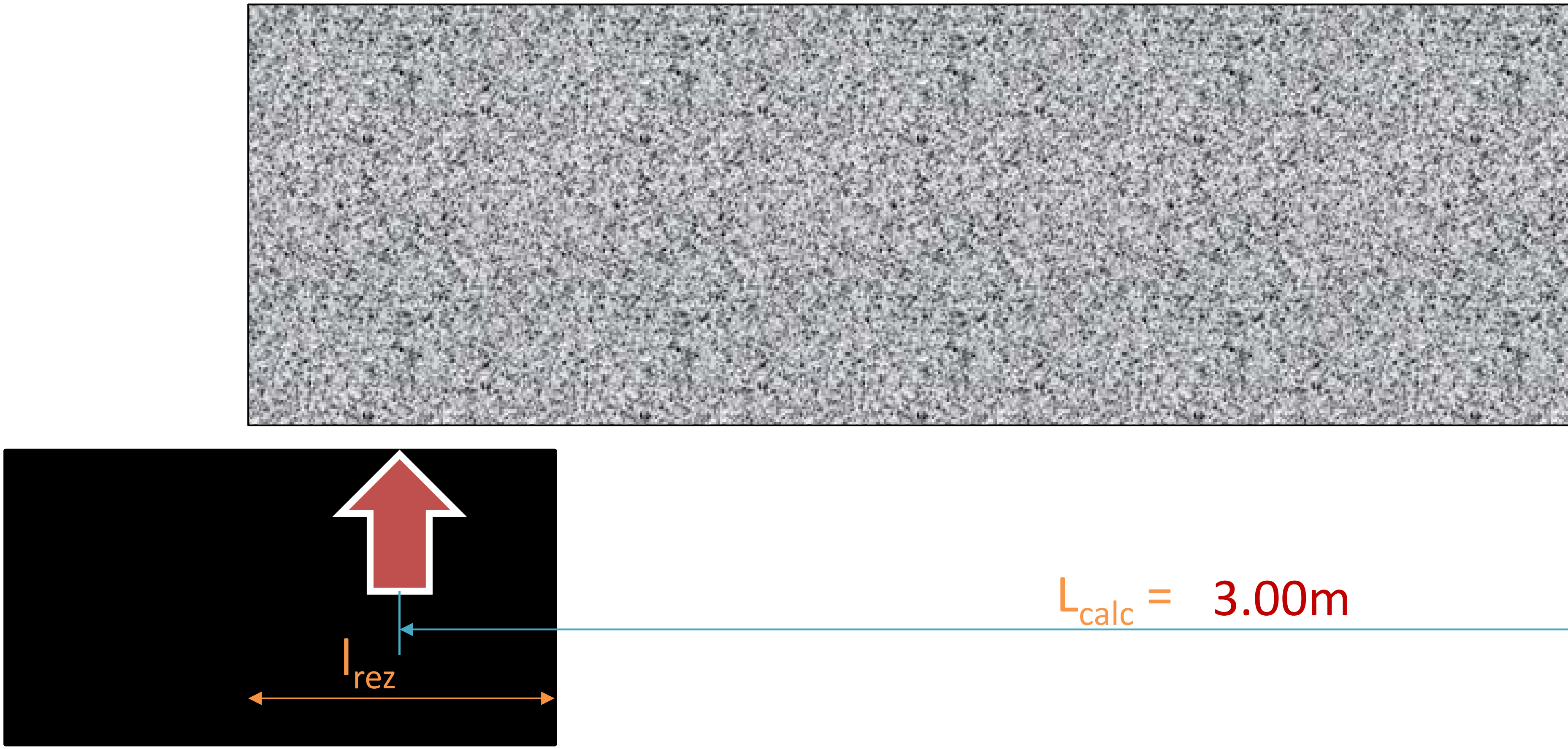
SUPPORT

Support → L_{calc}



SUPPORT

Support → L_{calc}

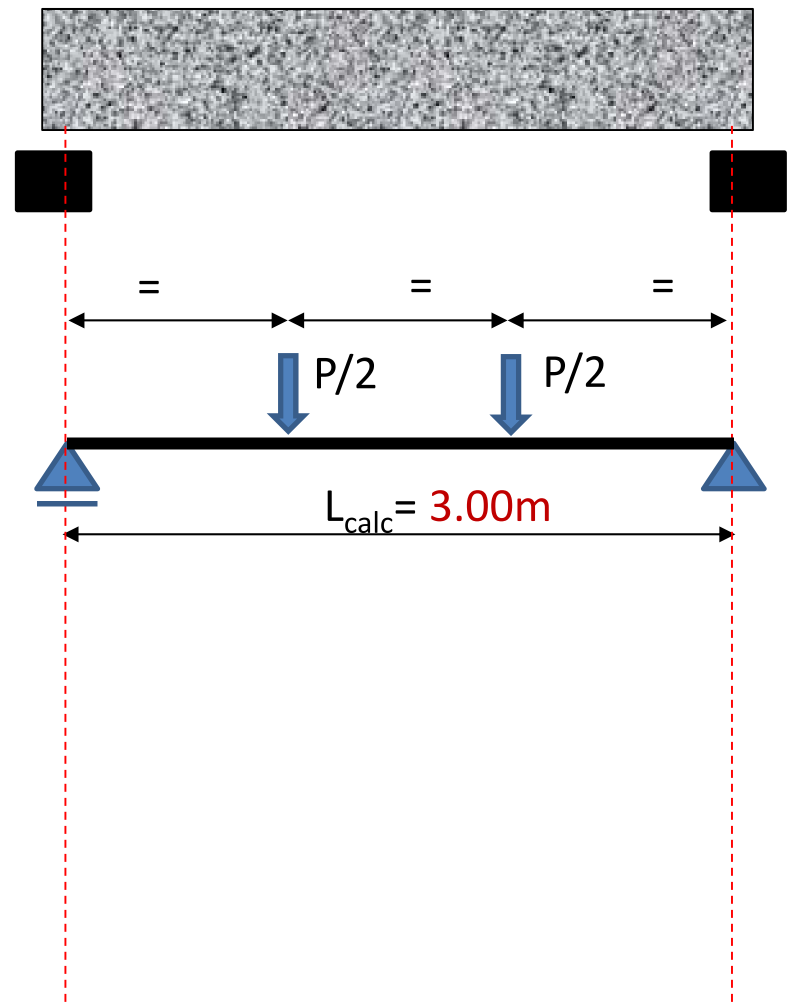


EFFORTS

Diagrams: M + V

M_{Ed} =

V_{Ed} =

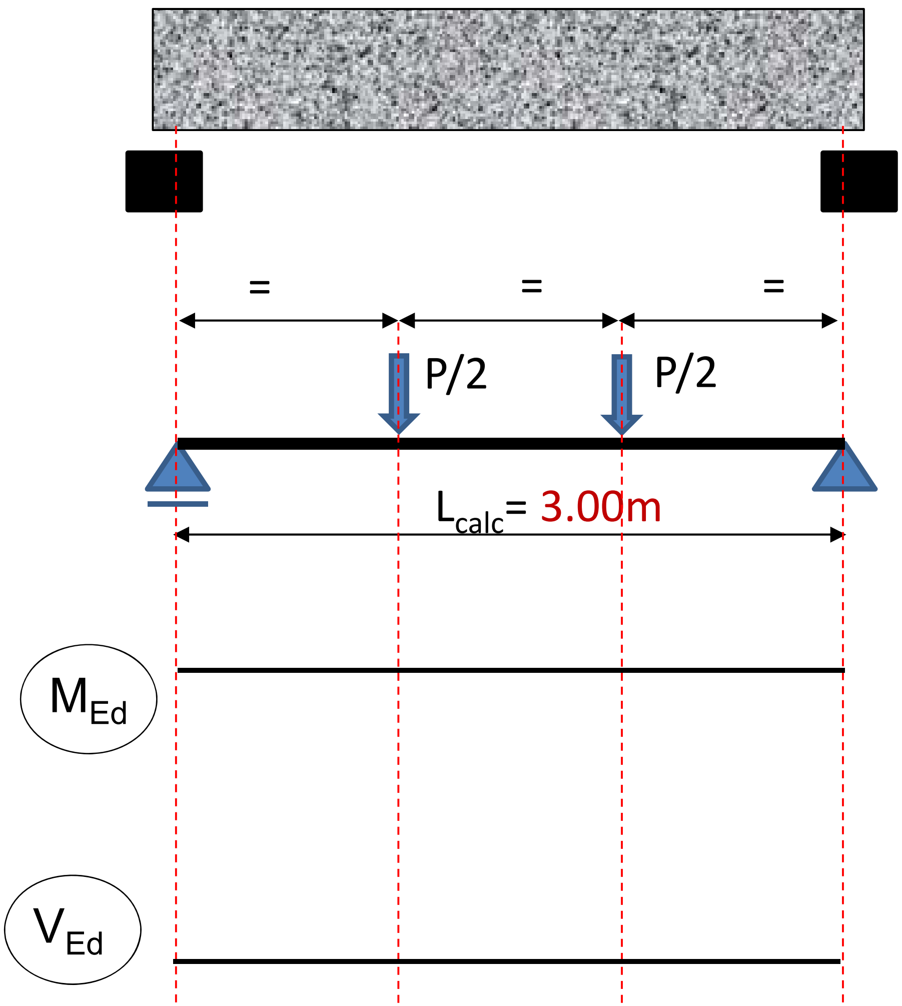


EFFORTS

Diagrams: M + V

M_{Ed} =

V_{Ed} =

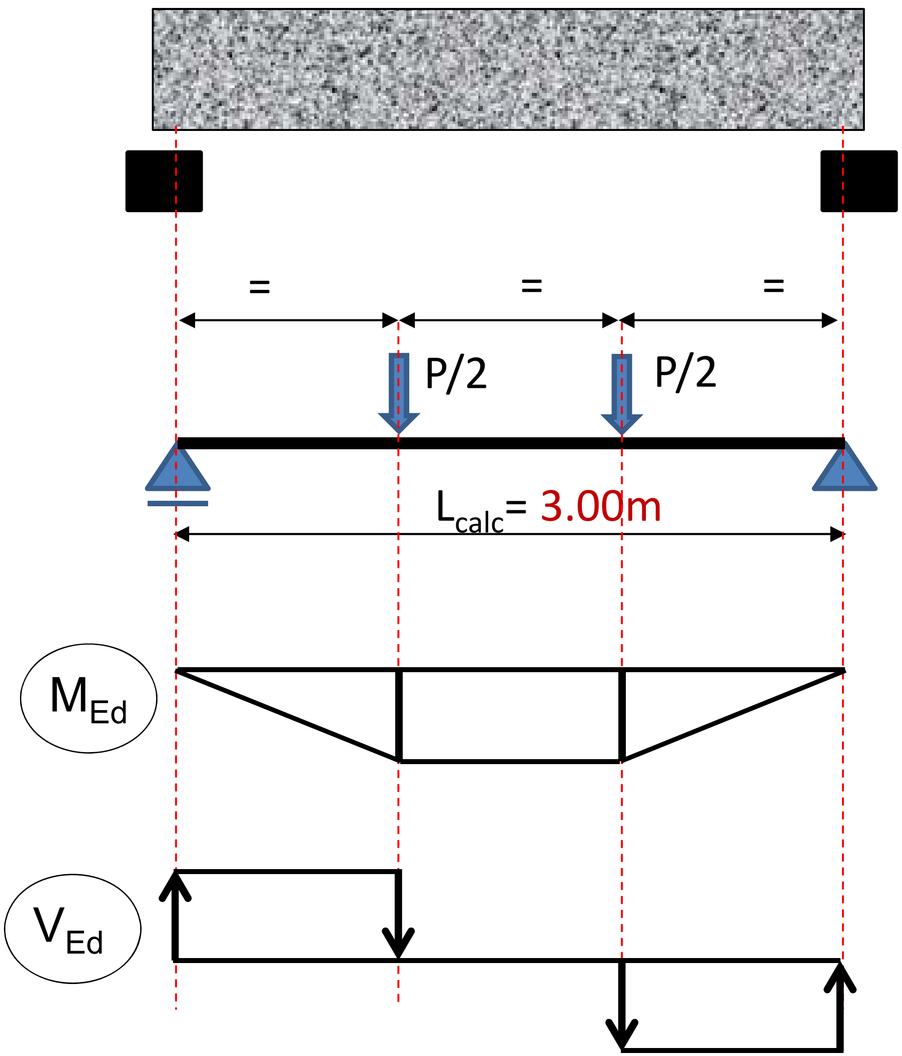


EFFORTS

Diagrams: M + V

M_{Ed} =

V_{Ed} =

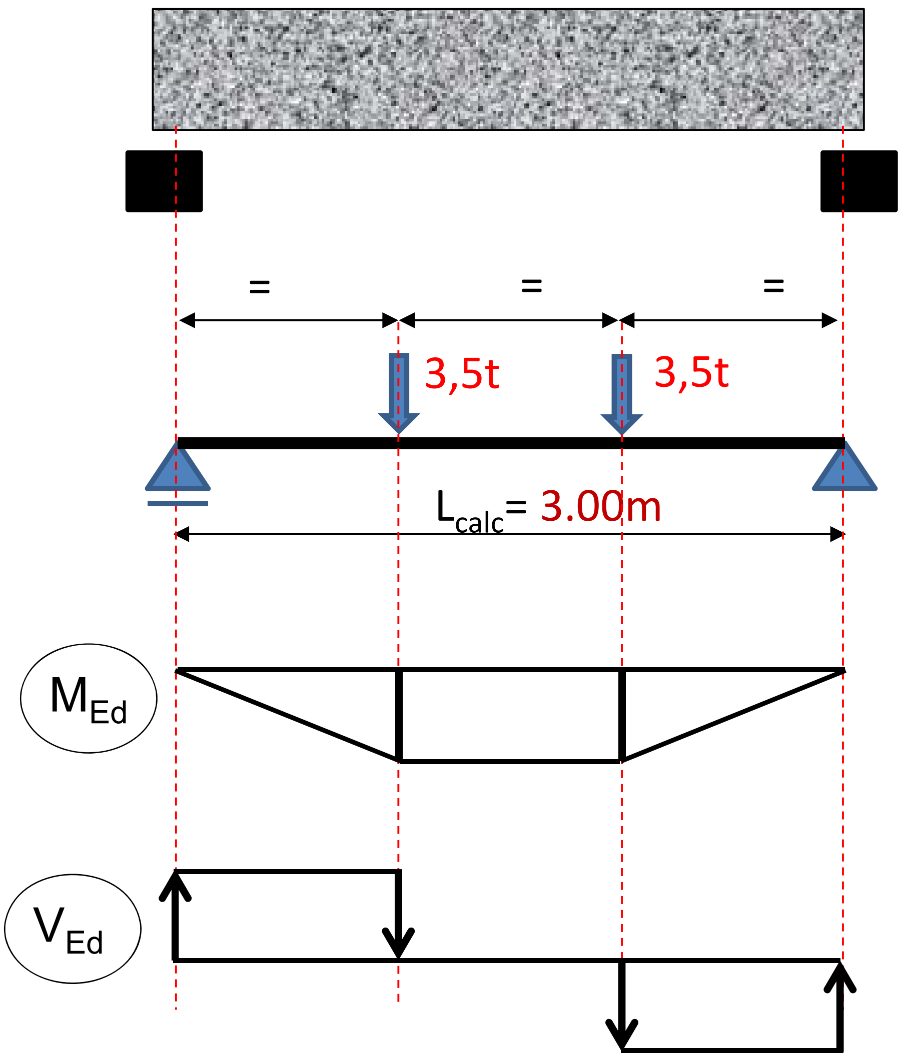


EFFORTS

Diagrams: M + V

M_{Ed} =

V_{Ed} =

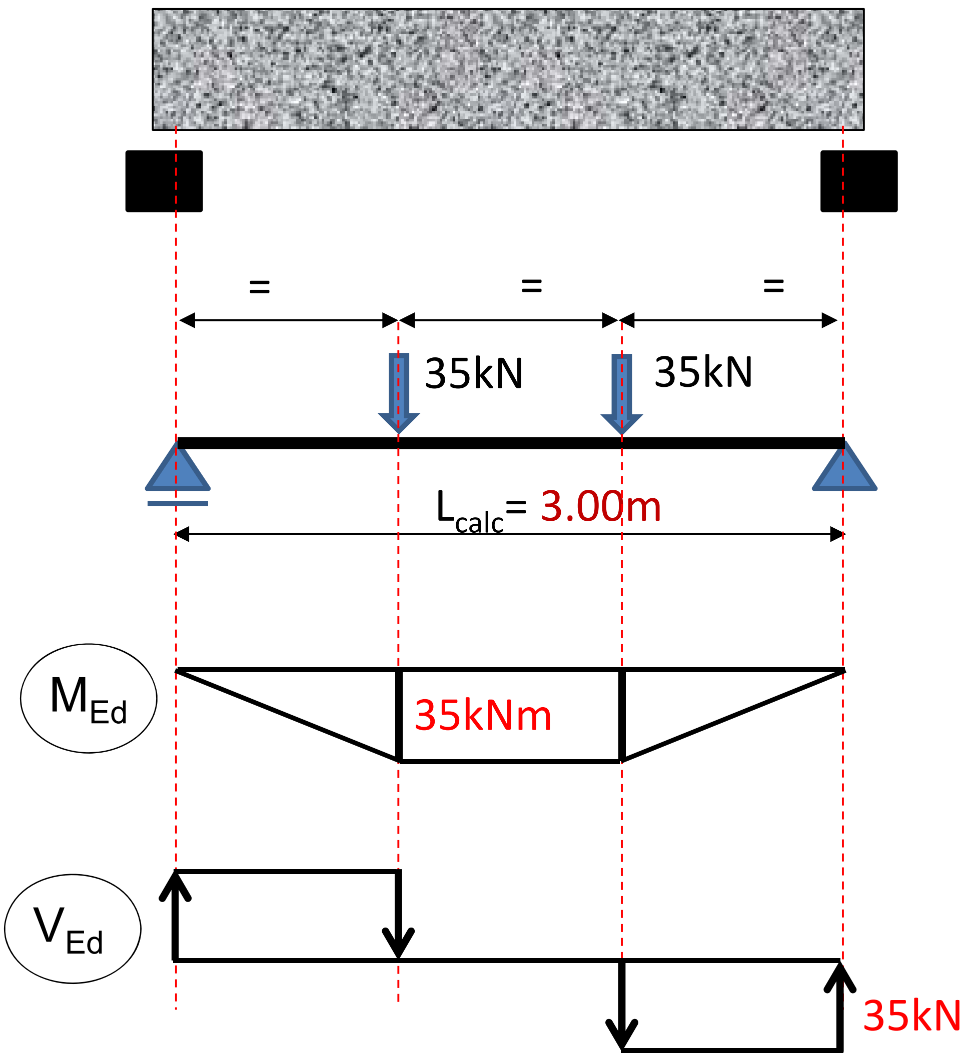


EFFORTS

Diagrams: M + V

M_{Ed} = 35 kNm

V_{Ed} = 35 kN



EFFORTS

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}}$$

$\mu \leq \mu_{lim}$!!!

$$\omega_s = 1 - \sqrt{1 - 2\mu}$$

$$p$$

d = ???

f_{cd} = ???

$$A_s = \omega_s b d \frac{f_{cd}}{f_{yd}}$$

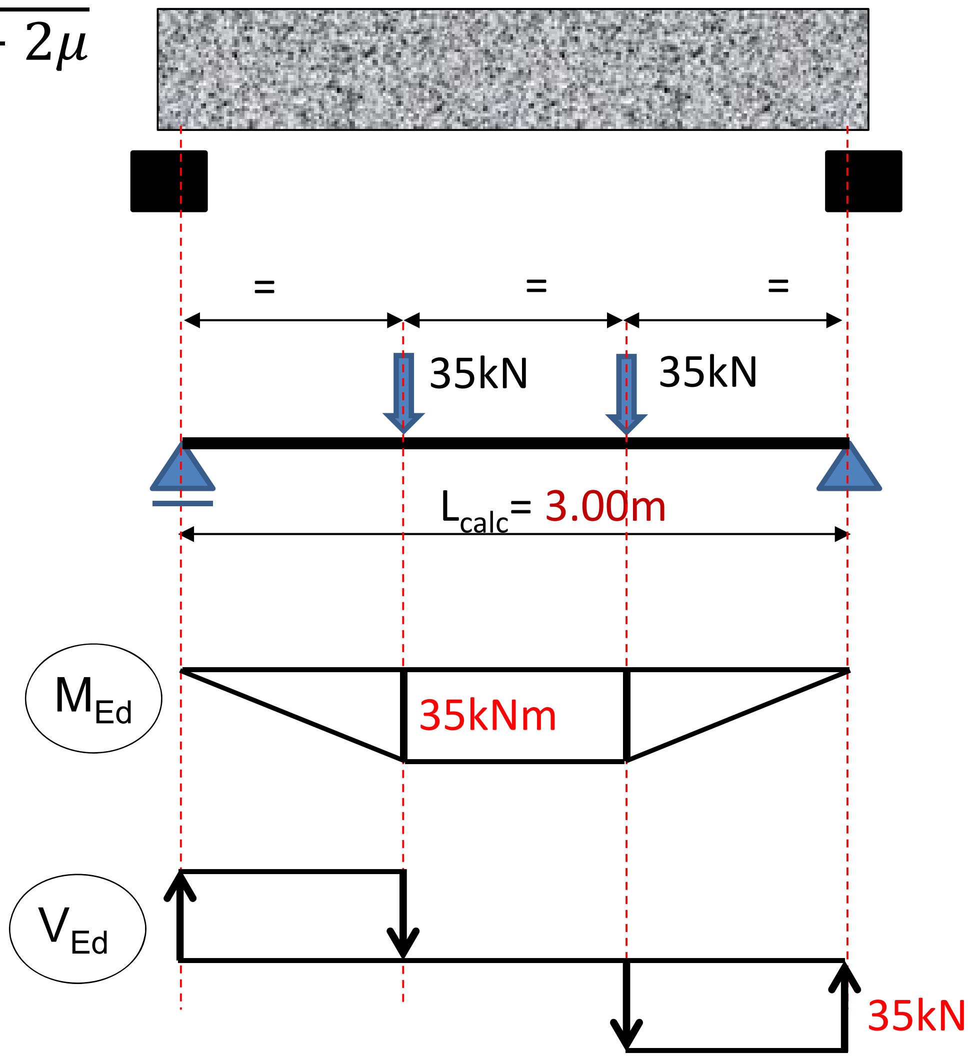
$$A_s = p \frac{bd}{100}$$

where $d = h - d_s$

$d_s = c_{nom} + \phi_s/2$

$\phi_s =$ 14…25 mm for beams

6…14 mm for slabs



CONCRETE COVER

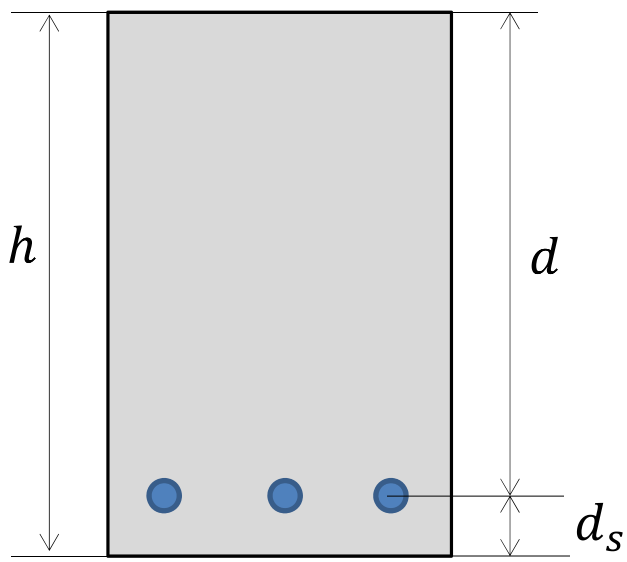
$d = h - d_s$

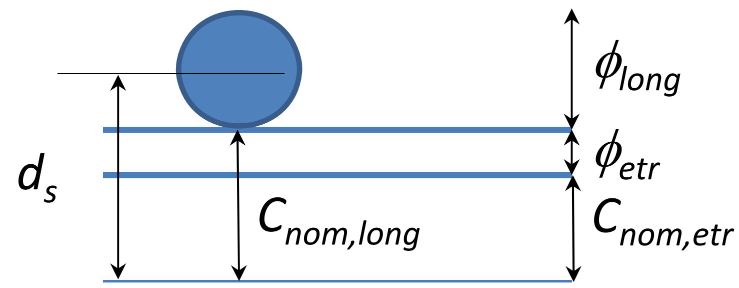
$d_s = c_{nom} + \phi_s/2$

$\phi_s =$ 14…25 mm for beams

6…14 mm for slabs

$$c_{nom} = c_{min} + \Delta c_{dev}$$





Δc_{dev} = 5 mm for slabs (A.N.)

= 10 mm rest of the elements (A.N.)

CONCRETE COVER

$$c_{nom} = c_{min} + \Delta c_{dev}$$

$$c_{min} = max\,\{\underbrace{c_{min,b}}_{\text{bond}};\, \underbrace{c_{min,dur}}_{\text{durability}};\; 10\,mm\}$$

CONCRETE COVER

$$c_{nom} = c_{min} + \Delta c_{dev}$$

$$c_{min} = max\,\{\underbrace{c_{min,b}}_{\text{bond}};\, c_{min,dur};\ 10\ mm\}$$

$$\boldsymbol{c_{min,b} \geq \phi}$$

CONCRETE COVER

$$c_{nom} = c_{min} + \Delta c_{dev}$$

$$c_{min} = max\,\{\underbrace{c_{min,b}}_{\textbf{bond}};\, c_{min,dur};\ 10\ mm\}$$

$c_{min,b} \geq \phi$ where ϕ = *14…25 mm for beams*

6…14 mm for slabs

ϕ_{long}	≈ 20 mm	longitudinal
ϕ_{etr}	≈ 8 mm	transversal

CONCRETE COVER

$$c_{nom} = c_{min} + \Delta c_{dev}$$

$$c_{min} = max\,\{c_{min,b}; \underbrace{c_{min,dur}}_{\textbf{durability}};\ 10\ mm\}$$

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	**Exposure Class according to Table 4.1**						
	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

CONCRETE COVER

$$c_{nom} = c_{min} + \Delta c_{dev}$$

$$c_{min} = max\,\{c_{min,b}; \underbrace{c_{min,dur}}_{\textbf{durability}};\ 10\ mm\}$$

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	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

CONCRETE COVER

Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class [1) 2)]	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

CONCRETE COVER

$$c_{nom} = c_{min} + \Delta c_{dev}$$

$$c_{min} = max\,\{c_{min,b};\, \underbrace{c_{min,dur}}_{\text{durability}};\; 10\,mm\}$$

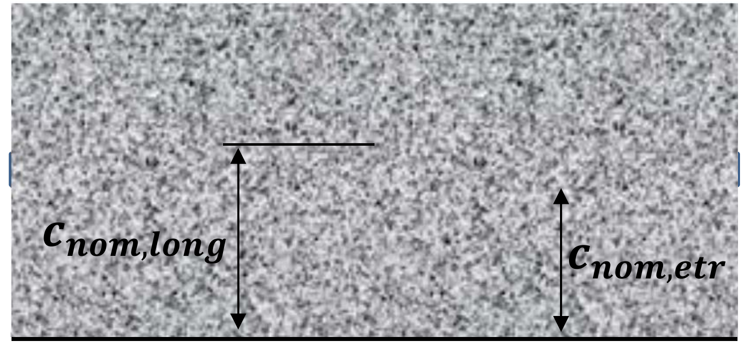
Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

$$\Rightarrow \boldsymbol{c_{min,dur} = 10\,mm}$$

CONCRETE COVER

$$c_{nom} = c_{min} + \Delta c_{dev}$$

LONGITUDINAL REINFORCEMENT	TRANSVERSAL REINF. (stirrup)
$c_{min} = max\left\{c_{min,b}; c_{min,dur}; 10\ mm\right\}$	
$= max\{20\ mm; 10\ mm; 10\ mm\}$	$= max\{8\ mm; 10\ mm; 10\ mm\}$
$c_{min,long} = 20$ mm	$c_{min,stir} = 10$ mm
$\Delta c_{dev} = 10\ mm$ (A.N.)	$\Delta c_{dev} = 10\ mm$ (A.N.)
$\boldsymbol{c_{nom,long} = 30\ mm}$	$\boldsymbol{c_{nom,stir} = 20\ mm}$
$\Rightarrow c_{nom,etr} = c_{nom,long} - \phi_{stir} = 22mm$	$\Rightarrow c_{nom,long} = c_{nom,stir} + \phi_{stir} = 28mm$
$c_{nom,etr} = 22mm > c_{nom,stir}^{nec} = 20\ mm$	$c_{nom,long} = 28mm < c_{nom,long}^{nec} = 30\ mm$
$\Rightarrow \boldsymbol{c_{nom,long} = 30\ mm}$ **OK!!!**	



$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} =$$

$\boldsymbol{d = h - d_s}$

$\boldsymbol{d_s = c_{nom} + \phi_s/2}$

$\phi_s = 20\ mm$

$d_s = c_{nom} + \phi_s/2 =$

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} =$$

$\boldsymbol{d = h - d_s}$

$\boldsymbol{d_s = c_{nom} + \phi_s/2}$

$\phi_s = 20\ mm$

$d_s = c_{nom} + \phi_s/2 =$ **40 mm**

$d = h - d_s =$

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} =$$

$d = h - d_s$

$d_s = c_{nom} + \phi_s/2$

$\phi_s = 20$ mm

$d_s = c_{nom} + \phi_s/2 =$ ***40 mm***

$d = h - d_s =$ **260 mm**

DESIGN STRENGTH

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} =$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c}$$

Situația de proiectare	γ_C (beton)	γ_S (oțel pentru beton armat)	γ_S (oțel pentru beton precomprimat)
Permanentă Tranzitorii	1,5	1,15	1,15
Accidentale	1,20	1,00	1,00

DESIGN STRENGTH

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} =$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c}$$ **= 13,33 N/mm²**

Table 2.1N: Partial factors for materials for ultimate limit states

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} = \mathbf{0.259}$$

CHECKING OF FAILURE MODE

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} = \mathbf{0.259}$$

$$\boldsymbol{\mu \leq \mu_{lim}???}$$

$$\mu_{lim} = 0.8\xi_{lim}(1 - 0.4\xi_{lim})$$

$$\xi_{lim} = \frac{3.5}{3.5 + 1000 f_{yd}/E_s}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s}$$

Table 2.1N: Partial factors for materials for ultimate limit states

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

CHECKING OF FAILURE MODE

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} = \mathbf{0.259}$$

$$\boldsymbol{\mu \le \mu_{lim}???}$$

$$\mu_{lim} = 0.8\xi_{lim}(1 - 0.4\xi_{lim})$$

$$\xi_{lim} = \frac{3.5}{3.5 + 1000 f_{yd}/E_s}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = 435\ \text{N/mm}^2$$

Table 2.1N: Partial factors for materials for ultimate limit states

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

CHECKING OF FAILURE MODE

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} = \mathbf{0.259}$$

$$\boldsymbol{\mu \le \mu_{lim}}???$$

$$\mu_{lim} = 0.8\xi_{lim}(1 - 0.4\xi_{lim})$$

$$\xi_{lim} = \frac{3.5}{3.5 + 1000 f_{yd}/E_s}$$

CHECKING OF FAILURE MODE

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} = \mathbf{0.259}$$

$$\boldsymbol{\mu \leq \mu_{lim}}???$$

$$\mu_{lim} = 0.8\xi_{lim}(1 - 0.4\xi_{lim})$$

$$\xi_{lim} = \frac{3.5}{3.5 + 1000 f_{yd}/E_s} = \mathbf{0{,}617}$$

CHECKING OF FAILURE MODE

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} = \mathbf{0.259}$$

$$\boldsymbol{\mu \leq \mu_{lim}???}$$

$$\mu_{lim} = 0.8\xi_{lim}(1 - 0.4\xi_{lim}) = \mathbf{0,372}$$

$$\xi_{lim} = \frac{3.5}{3.5 + 1000 f_{yd}/E_s} = \mathbf{0,617}$$

CHECKING OF FAILURE MODE

$$\mu = \frac{M_{Ed}}{bd^2 f_{cd}} = \mathbf{0.259}$$

$$\boldsymbol{\mu \leq \mu_{lim}???}$$

$$\mu_{lim} = 0.8\xi_{lim}(1 - 0.4\xi_{lim}) = \mathbf{0,372}$$

$$\xi_{lim} = \frac{3.5}{3.5 + 1000 f_{yd}/E_s} = \mathbf{0,617}$$

$$\boldsymbol{\mu \leq \mu_{lim}} \ :)$$

DESIGN OF LONGITUDINAL REINFORCEMENT

$$\omega_s = 1 - \sqrt{1 - 2\mu}$$

$$A_s = \omega_s b d \frac{f_{cd}}{f_{yd}}$$

DESIGN OF LONGITUDINAL REINFORCEMENT

$$\omega_s = 1 - \sqrt{1 - 2\mu}$$ **= 0,306**

$$A_s = \omega_s b d \frac{f_{cd}}{f_{yd}}$$

DESIGN OF LONGITUDINAL REINFORCEMENT

$$\omega_s = 1 - \sqrt{1 - 2\mu}$$ **= 0,306**

$$A_s = \omega_s b d \frac{f_{cd}}{f_{yd}}$$ **= 365 mm²**

$\Rightarrow$ REINFORCEMENT PROPOSAL ?

DESIGN OF LONGITUDINAL REINFORCEMENT

Diametrul mm	Aria secțiunii transversale pentru *n* bare, în cm^2										Masa kg/m
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	
6	0,283	0,570	0,850	1,130	1,420	1,700	1,980	2,260	2,550	2,830	0,222
8	0,503	1,010	1,510	2,010	2,510	3,020	3,520	4,020	4,530	5,030	0,395
10	0,785	1,570	2,350	3,140	3,920	4,710	5,490	6,280	7,060	7,850	0,617
12	1,130	2,260	3,390	4,520	5,650	6,780	7,910	9,040	10,17	11,30	0,888
14	1,540	3,080	4,620	6,160	7,700	9,240	10,78	12,32	13,86	15,40	1,120
16	2,010	4,020	6,030	8,040	10,05	12,06	14,07	16,08	18,09	20,10	1,580
18	2,540	5,080	7,620	10,16	12,70	15,24	17,78	20,32	22,86	25,40	1,990
20	3,140	6,280	9,420	12,56	15,70	18,84	21,98	25,12	28,26	31,40	2,460
22	3,800	7,600	11,40	15,20	19,00	22,80	26,60	30,40	34,20	38,00	2,980
25	4,910	9,820	14,73	19,64	24,55	29,46	34,37	39,28	44,19	49,10	3,850
28	6,160	12,32	18,48	24,64	30,80	36,96	43,12	49,28	55,44	61,60	4,840
32	8,040	16,08	24,12	32,16	40,20	48,24	56,28	64,32	72,36	80,40	6,310
36	10,20	20,40	30,60	40,80	51,00	61,20	71,40	81,60	91,80	102,00	7,990
40	12,60	25,20	37,80	50,40	63,00	75,60	88,20	100,80	113,40	126,00	9,870

DESIGN OF LONGITUDINAL REINFORCEMENT

$$\omega_s = 1 - \sqrt{1 - 2\mu}$$ **= 0,306**

$$A_s = \omega_s b d \frac{f_{cd}}{f_{yd}}$$ **= 365 mm^2**

$\Rightarrow 2\ \phi\ 16$

$\Rightarrow 3\ \phi\ 14$

DESIGN OF LONGITUDINAL REINFORCEMENT

$$\Rightarrow 2\ \phi\ 16$$

$$d_s = c_{nom} + \phi_s/2 =$$

$$d = h - d_s =$$

$$A_{s,min} = 0{,}26 \frac{f_{ctm}}{f_{yk}} bd$$

$$A_{s,min} \geq 0{,}0013 bd$$

$$A_{s,max} = 0{,}04 A_c$$

DESIGN OF LONGITUDINAL REINFORCEMENT

$\Rightarrow 2\ \phi\ 16$ = 402 mm²

$d_s = c_{nom} + \phi_s/2 =$ 38 mm

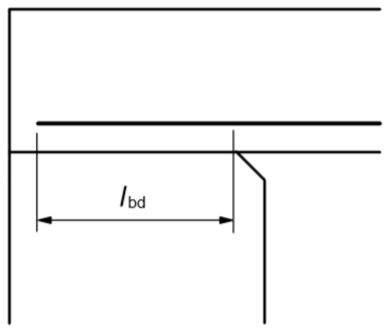
$d = h - d_s =$ 262 mm

$$A_{s,min} = 0{,}26\frac{f_{ctm}}{f_{yk}}bd$$ = 45 mm² **ok!**

$$A_{s,min} \geq 0{,}0013bd$$ = 51 mm² **ok!**

$$A_{s,max} = 0{,}04A_c$$ = 1800 mm² **ok!**

DESIGN OF ANCHORAGE LENGTH



a) Direct support: Beam supported by wall or column

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \geq l_{b,min}$$

$$l_{b,rqd} = \frac{\phi \cdot f_{yd}}{4 \cdot f_{bd}}$$

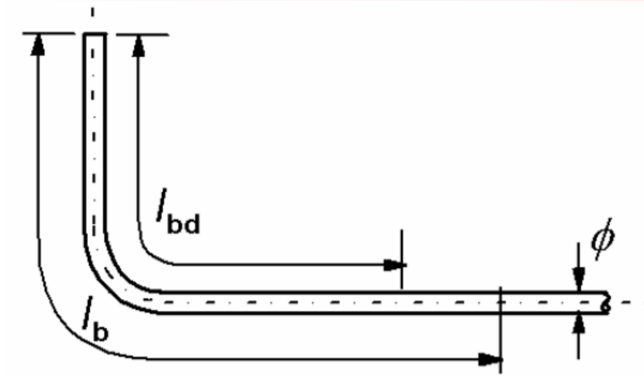
$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd}$$

DESIGN OF ANCHORAGE LENGTH

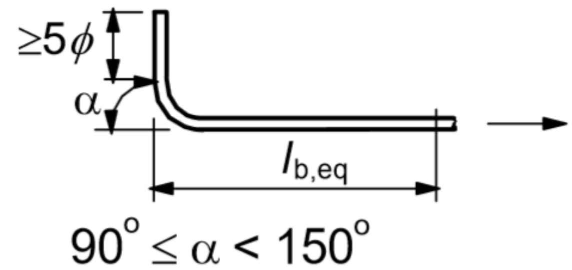
Table 8.2: Values of α_1, α_2, α_3, α_4 and α_5 coefficients

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Shape of bars	Straight	$\alpha_1 = 1,0$	$\alpha_1 = 1,0$
	Other than straight (see Figure 8.1 (b), (c) and (d)	$\alpha_1 = 0,7$ if $c_d > 3\phi$ otherwise $\alpha_1 = 1,0$ (see Figure 8.3 for values of c_d)	$\alpha_1 = 1,0$
Concrete cover	Straight	$\alpha_2 = 1 - 0,15 (c_d - \phi)/\phi$ $\geq 0,7$ $\leq 1,0$	$\alpha_2 = 1,0$
	Other than straight (see Figure 8.1 (b), (c) and (d))	$\alpha_2 = 1 - 0,15 (c_d - 3\phi)/\phi$ $\geq 0,7$ $\leq 1,0$ (see Figure 8.3 for values of c_d)	$\alpha_2 = 1,0$
Confinement by transverse reinforcement not welded to main reinforcement	All types	$\alpha_3 = 1 - K\lambda$ $\geq 0,7$ $\leq 1,0$	$\alpha_3 = 1,0$
Confinement by welded transverse reinforcement*	All types, position and size as specified in Figure 8.1 (e)	$\alpha_4 = 0,7$	$\alpha_4 = 0,7$
Confinement by transverse pressure	All types	$\alpha_5 = 1 - 0,04p$ $\geq 0,7$ $\leq 1,0$	-

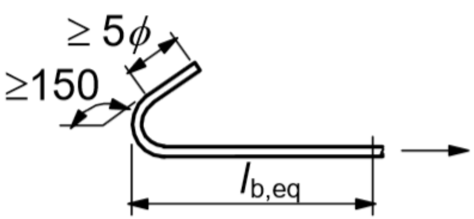
DESIGN OF ANCHORAGE LENGTH



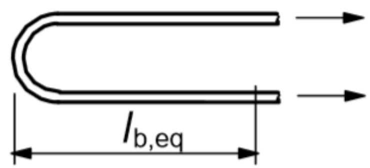
a) Basic tension anchorage length, l_b, for any shape measured along the centreline



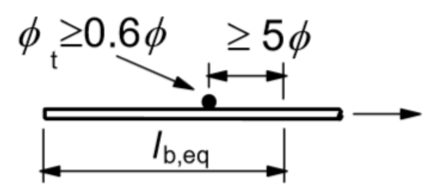
b) Equivalent anchorage length for standard bend



c) Equivalent anchorage length for standard hook



d) Equivalent anchorage length for standard loop



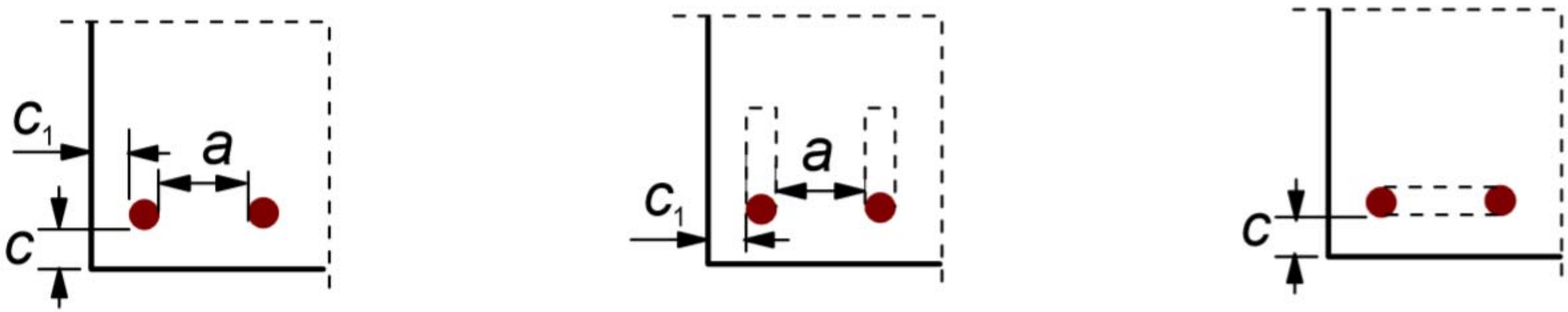
e) Equivalent anchorage length for welded transverse bar

Figure 8.1: Methods of anchorage other than by a straight bar

DESIGN OF ANCHORAGE LENGTH

Table 8.2: Values of α_1, α_2, α_3, α_4 and α_5 coefficients

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Shape of bars	Straight	$\alpha_1 = 1,0$	$\alpha_1 = 1,0$
	Other than straight (see Figure 8.1 (b), (c) and (d)	$\alpha_1 = 0,7$ if $c_d > 3\phi$ otherwise $\alpha_1 = 1,0$ (see Figure 8.3 for values of c_d)	$\alpha_1 = 1,0$



a) Straight bars
$c_d = \min(a/2, c_1, c)$

b) Bent or hooked bars
$c_d = \min(a/2, c_1)$

c) Looped bars
$c_d = c$

Figure 8.3: Values of c_d for beams and slabs

$$c_d = \min(58/2\,;30) = 29mm$$

DESIGN OF ANCHORAGE LENGTH

Table 8.2: Values of α_1, α_2, α_3, α_4 and α_5 coefficients

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Shape of bars	Straight	$\alpha_1 = 1,0$	$\alpha_1 = 1,0$
	Other than straight (see Figure 8.1 (b), (c) and (d)	$\alpha_1 = 0,7$ if $c_d > 3\phi$ otherwise $\alpha_1 = 1,0$ (see Figure 8.3 for values of c_d)	$\alpha_1 = 1,0$

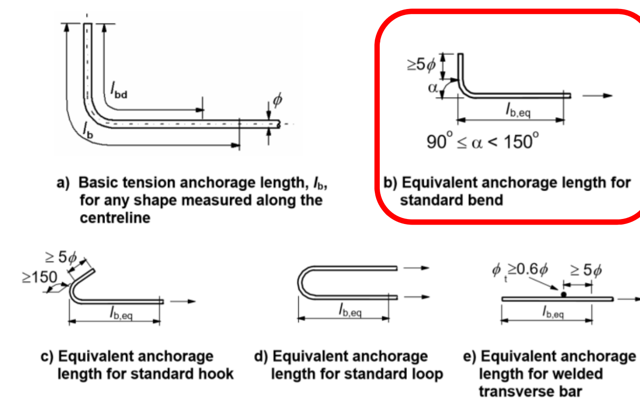
(2) As a simplified alternative to 8.4.4 (1) the tension anchorage of certain shapes shown in Figure 8.1 may be provided as an equivalent anchorage length, $l_{b,eq}$. $l_{b,eq}$ is defined in this figure and may be taken as:

- $\alpha_1 l_{b,rqd}$ for shapes shown in Figure 8.1b to 8.1d (see Table 8.2 for values of α_1)
- $\alpha_4 l_{b,rqd}$ for shapes shown in Figure 8.1e (see Table 8.2 for values of α_4).

where

α_1 and α_4 are defined in (1)

$l_{b,rqd}$ is calculated from Expression (8.3)



a) Basic tension anchorage length, l_b, for any shape measured along the centreline

b) Equivalent anchorage length for standard bend

c) Equivalent anchorage length for standard hook

d) Equivalent anchorage length for standard loop

e) Equivalent anchorage length for welded transverse bar

Figure 8.1: Methods of anchorage other than by a straight bar

DESIGN OF ANCHORAGE LENGTH

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd}$$

→ design value of the ultimate bond stress

where:

η_1 - is a coefficient related to the quality of the bond condition and the position of the bar during concreting (see Figure 8.2):
= 1,0 when 'good' conditions are obtained and
= 0,7 for all other cases and for bars in structural elements built with slip-forms, unless it can be shown that 'good' bond conditions exist

η_2 - is related to the bar diameter:
= 1,0 for ϕ ≤ 32 mm
= (132 - ϕ)/100 for ϕ > 32 mm

$$f_{ctd} = \frac{f_{ctk0,05}}{\gamma_c}$$

$$f_{bd} =$$

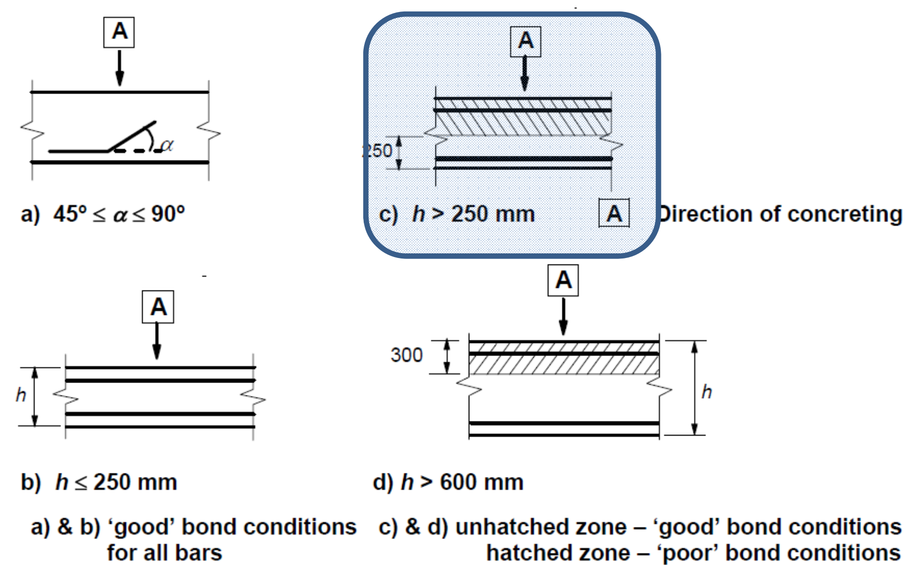


Figure 8.2: Description of bond conditions

DESIGN OF ANCHORAGE LENGTH

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd}$$ → design value of the ultimate bond stress

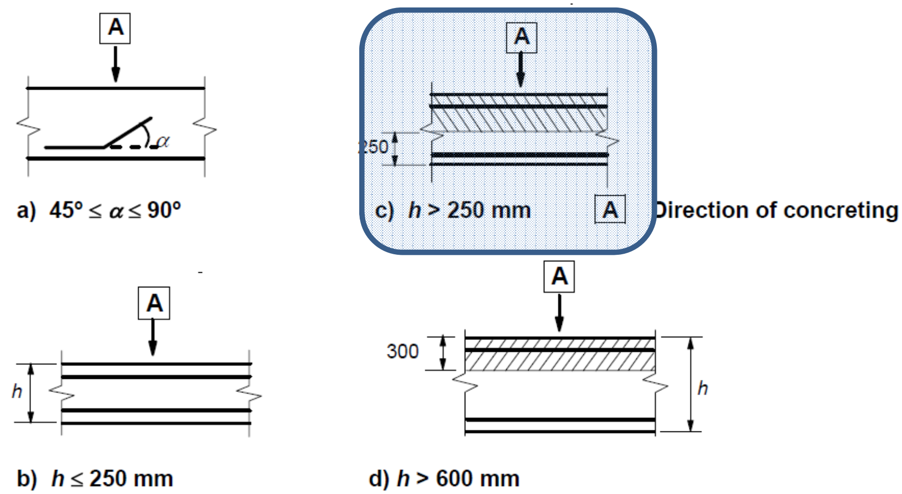
where:

η_1 - is a coefficient related to the quality of the bond condition and the position of the bar during concreting (see Figure 8.2):
= 1,0 when 'good' conditions are obtained and
= 0,7 for all other cases and for bars in structural elements built with slip-forms, unless it can be shown that 'good' bond conditions exist

η_2 - is related to the bar diameter:
= 1,0 for ϕ ≤ 32 mm
= (132 - ϕ)/100 for ϕ > 32 mm

$$f_{ctd} = \frac{f_{ctk0,05}}{\gamma_c}$$ **= 1,00 N/mm²**

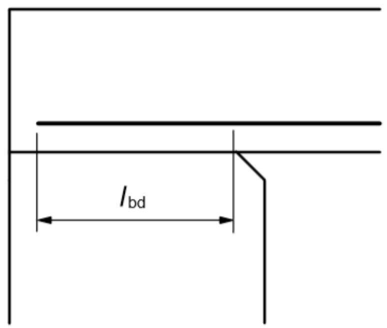
$$f_{bd} =$$ **2,25 N/mm²**



a) & b) 'good' bond conditions for all bars
c) & d) unhatched zone – 'good' bond conditions; hatched zone – 'poor' bond conditions

Figure 8.2: Description of bond conditions

DESIGN OF ANCHORAGE LENGTH



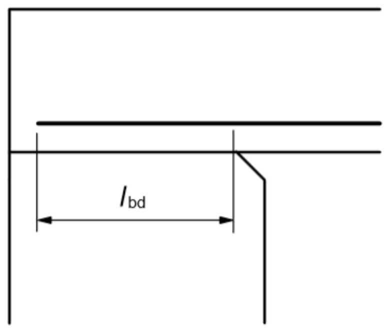
a) Direct support: Beam supported by wall or column

$$f_{bd} =$$

$$l_{b,rqd} = \frac{\phi \cdot f_{yd}}{4 \cdot f_{bd}}$$

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \geq l_{b,min}$$

DESIGN OF ANCHORAGE LENGTH



a) Direct support: Beam supported by wall or column

$$f_{bd} = \quad 2{,}25\ \text{N/mm}^2$$

$$l_{b,rqd} = \frac{\phi \cdot f_{yd}}{4 \cdot f_{bd}} \quad =773\ \text{mm}$$

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \geq l_{b,min} \quad =773\ \text{mm}$$

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix} \left[C_{Rd,c} k (100\rho_l f_{ck})^{1/3} + k_1 \sigma_{cp}\right] b_w d \\ (v_{min} + k_1 \sigma_{cp}) b_w d \end{pmatrix}$$

$C_{Rd,c} = 0{,}18/\gamma_c =$

$k = 1 + \sqrt{\frac{200}{d}} =$

$k \leq 2$

$k_1 = 0{,}15$ $\qquad A.N.$

$\sigma_{cp} = N_{Ed}/A_c < 0{,}2 f_{cd}$

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix} \left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \\ (v_{min} + k_1\sigma_{cp})b_w d \end{pmatrix}$$

$C_{Rd,c} = 0{,}18/\gamma_c =$ 0.12

$k = 1 + \sqrt{\frac{200}{d}} =$ 1.877

$k \leq 2$ √

$k_1 = 0{,}15$ $A.N.$

$\sigma_{cp} = N_{Ed}/A_c < 0{,}2 f_{cd}$ =0

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix} \left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \\ (v_{min} + k_1\sigma_{cp})b_w d \end{pmatrix}$$

$$\rho_l = \frac{A_{sl}}{b_w d} =$$

$$\rho_l \leq 0.02$$

A_{sl} = ???

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix}\left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \\ (v_{min} + k_1\sigma_{cp})b_w d\end{pmatrix}$$

$$\rho_l = \frac{A_{sl}}{b_w d} =$$

$$\rho_l \leq 0.02$$

A_{sl} = ???

A_{sl} - is the area of the tensile reinforcement, which extends ≥ (l_{bd} + d) beyond the section considered

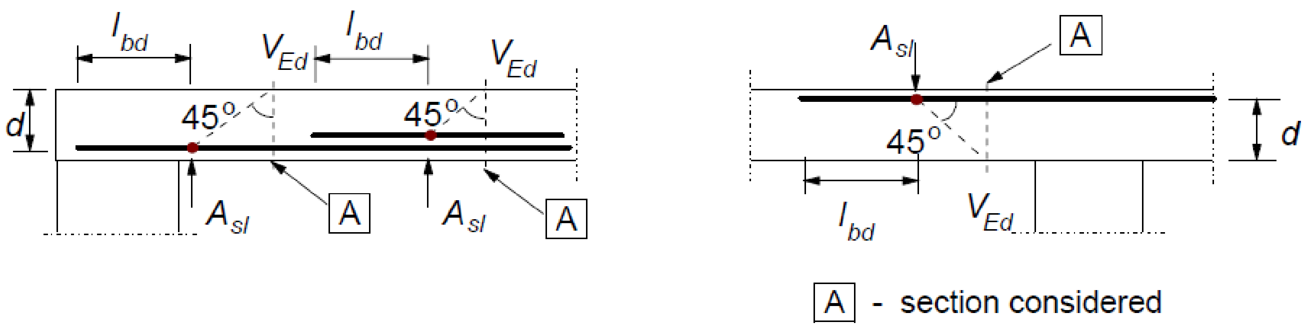


Figure 6.3: Definition of A_{sl} in Expression (6.2)

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix}\left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \\ (v_{min} + k_1\sigma_{cp})b_w d\end{pmatrix}$$

$$\rho_l = \frac{A_{sl}}{b_w d} = \quad 0.01$$

$A_{sl} = 2\phi16 = 402\ mm^2$

$$\rho_l \leq 0.02 \quad \surd$$

A_{sl} - is the area of the tensile reinforcement, which extends ≥ (l_{bd} + d) beyond the section considered

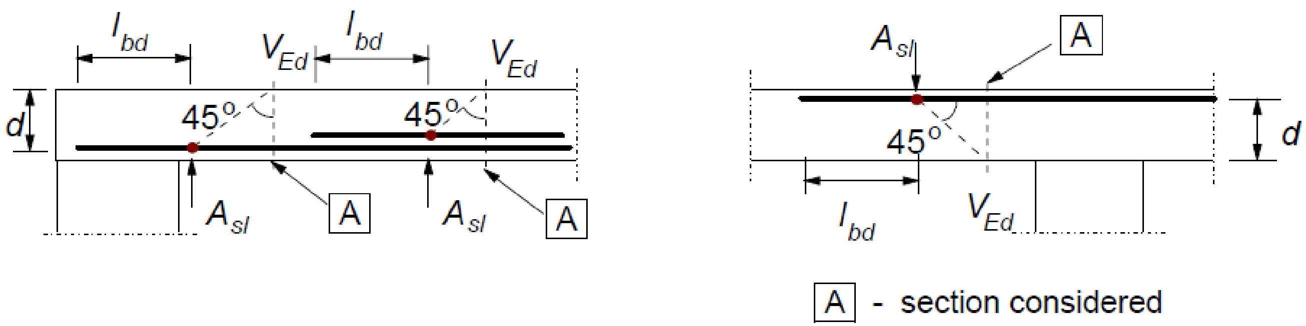


Figure 6.3: Definition of A_{sl} in Expression (6.2)

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix}\left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \\ (v_{min} + k_1\sigma_{cp})b_w d\end{pmatrix}$$

$$v_{min} = 0{,}035k^{3/2} \cdot f_{ck}^{1/2} =$$

$$V_{Rd,c} =$$

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix} \left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \\ (v_{min} + k_1\sigma_{cp})b_w d \end{pmatrix}$$

$v_{min} = 0{,}035k^{3/2} \cdot f_{ck}^{1/2} =$ 0.403

$V_{Rd,c} =$ 24.03 kN

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix} \overbrace{\left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d}^{24.03\ \text{kN}} \\ \underbrace{(v_{min} + k_1\sigma_{cp})b_w d}_{15.72\ \text{kN}} \end{pmatrix}$$

$v_{min} = 0{,}035k^{3/2} \cdot f_{ck}^{1/2} =$ 0.403

$V_{Rd,c} =$ 24.03 kN

DESIGN FOR SHEAR

$$V_{Rd,c} = 24.03\ kN \quad < \quad V_{Ed} = 35\ kN$$

→

DESIGN FOR SHEAR

$$V_{Rd,c} = 24.03\ kN \quad < \quad V_{Ed} = 35\ kN$$

→ **SHEAR REINFORCEMENT IS REQUIRED**

$$V_{Rd} = \min(V_{Rd,s}; V_{Rd,max})$$

DESIGN FOR SHEAR

$$V_{Rd,c} = 24.03\ kN \quad < \quad V_{Ed} = 35\ kN$$

→ SHEAR REINFORCEMENT IS REQUIRED

$$V_{Rd} = \min(V_{Rd,s}; V_{Rd,max})$$

DESIGN FOR SHEAR

CAPACITY OF THE COMPRESSED (CONCRETE) CHORD ($V_{Rd,max}$)

$$V_{Rd,max} = \alpha_{cw} b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (tg\theta + ctg\theta)$$

α_{cw} - is a coefficient taking account of the state of the stress in the compression chord

$\alpha_{cw} = 1$ for RC elements

$\alpha_{cw} > 1$ for PC elements

ν_1 - is a strength reduction factor for concrete cracked in shear

$$\nu_1 = \nu = 0,6\left(1 - \frac{f_{ck}}{250}\right) =$$

(A.N.)

$z \approx 0.9d =$

DESIGN FOR SHEAR

CAPACITY OF THE COMPRESSED (CONCRETE) CHORD ($V_{Rd,max}$)

$$V_{Rd,max} = \alpha_{cw} b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (tg\theta + ctg\theta)$$

α_{cw} - is a coefficient taking account of the state of the stress in the compression chord

$\alpha_{cw}=1$ for RC elements

$\alpha_{cw}>1$ for PC elements

ν_1 - is a strength reduction factor for concrete cracked in shear

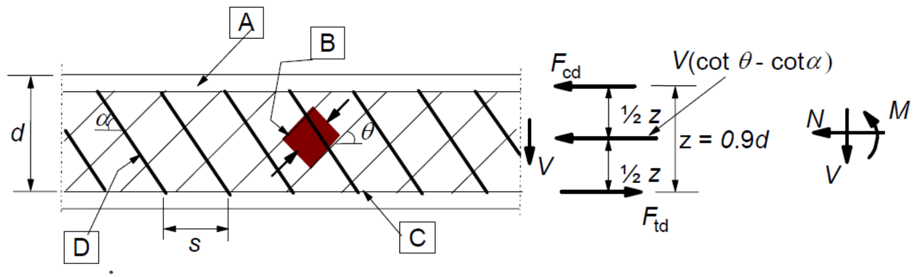
$$\nu_1 = \nu = 0{,}6\left(1 - \frac{f_{ck}}{250}\right) =$$ 0.552 (A.N.)

$z \approx 0.9d =$ 236 mm

DESIGN FOR SHEAR

CAPACITY OF THE COMPRESSED (CONCRETE) CHORD ($V_{Rd,max}$)

$$V_{Rd,max} = \alpha_{cw} b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (tg\theta + ctg\theta)$$



A - compression chord, B - struts, C - tensile chord, D - shear reinforcement

Choosing

$\theta = 45°$ $\alpha = 90°$

$V_{Rd,max} =$

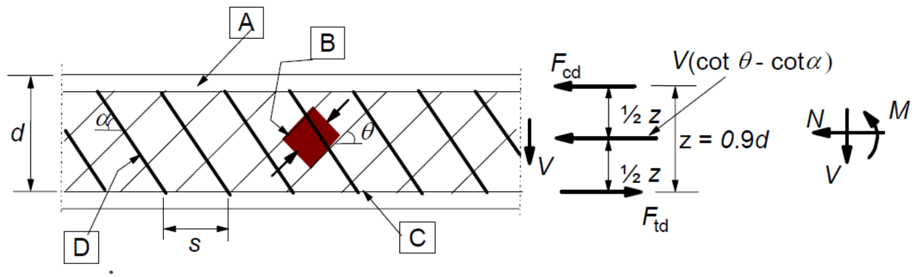
DESIGN FOR SHEAR

CAPACITY OF THE COMPRESSED (CONCRETE) CHORD ($V_{Rd,max}$)

$$V_{Rd,max} = \alpha_{cw} b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (tg\theta + ctg\theta)$$

Choosing

$\theta = 45°$ $\alpha = 90°$



A - compression chord, B - struts, C - tensile chord, D - shear reinforcement

$V_{Rd,max} =$ **130.16 kN**

DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

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

Choosing

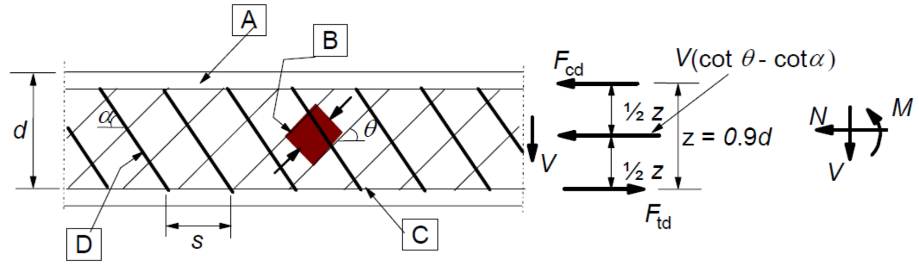
A_{sw} - the cross-sectional area of the shear reinforcement

or

s - the spacing of the stirrups

and

$\theta = 45°$ $\alpha = 90°$



A - compression chord, B - struts, C - tensile chord, D - shear reinforcement

From condition $V_{Rd,s} = V_{Ed}$ $\Rightarrow$ $\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot ctg\theta}$

DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

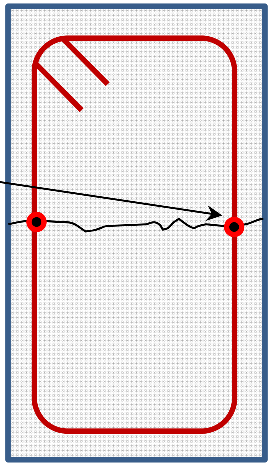
$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot ctg\theta} =$$

Diametre of stirrups are imposed (ϕ)

$\Rightarrow$ ***$A_{sw}=nA_s$***

(n= no shear arms!!!)

$\Rightarrow$ ***s*** *(spacing)*



DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

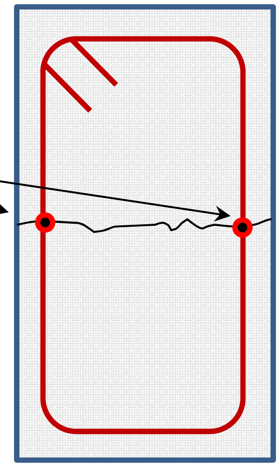
$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot ctg\theta} =$$

Diametre of stirrups are imposed (ϕ)

⇒ $A_{sw} = nA_s$

(n= no shear arms!!!)

⇒ **s** *(spacing)*



$$A_{sw} = 2\phi 6 \ ?$$

$$= 2\phi 8 \ ?$$

DESIGN FOR SHEAR

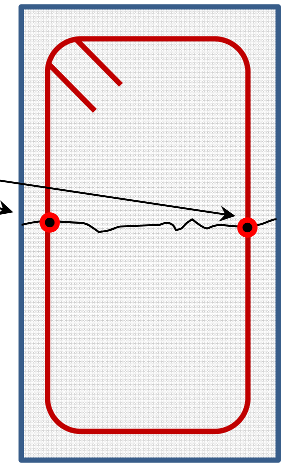
CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot ctg\theta} =$$

Diametre of stirrups are imposed (ϕ)

⇒ $A_{sw}=nA_s$
(n= no shear arms!!!)

⇒ **s** *(spacing)*



$$A_{sw} = 2\phi 6 = \quad 56.5\ \text{mm}^2$$

DESIGN FOR SHEAR

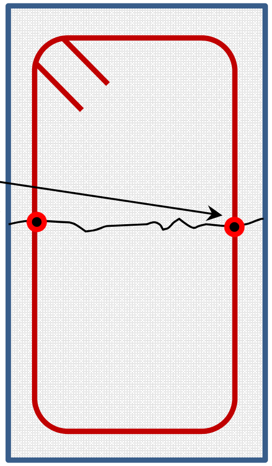
CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot ctg\theta} =$$ 0.341 mm

Diametre of stirrups are imposed (ϕ)

$\Rightarrow$ $A_{sw}=nA_s$
(n= no shear arms!!!)

$\Rightarrow$ **s** (spacing)



$$A_{sw} = 2\phi 6 =$$ 56.5 mm²

DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

$A_{sw} = 2\phi 6 =$ 56.5 mm^2

Reinforcement ratio for shear reinforcement

$$\rho_{sw} = \frac{A_{sw}}{b \cdot d \cdot sin\alpha} =$$

$$\rho_{sw,min} = (0{,}08\sqrt{f_{ck}})/f_{yk} =$$

ρ_{sw} **>(?)** $\rho_{sw,min}$

DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

$A_{sw} = 2\phi 6 =$ 56.5 mm^2

Reinforcement ratio for shear reinforcement

$$\rho_{sw} = \frac{A_{sw}}{b \cdot d \cdot sin\alpha} =$$ 1.439x10^{-3}

$$\rho_{sw,min} = (0{,}08\sqrt{f_{ck}})/f_{yk} =$$ 0.715x10^{-3}

ρ_{sw} **>(?)** $\rho_{sw,min}$

√

DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot ctg\theta} =$$ 0.341 mm

$$A_{sw} = 2\phi 6 =$$ 56.5 mm^2

$\Rightarrow$ **s =**

DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot ctg\theta} =$$ 0.341 mm

$$A_{sw} = 2\phi 6 =$$ 56.5 mm²

$\Rightarrow$ **s =** 166 mm

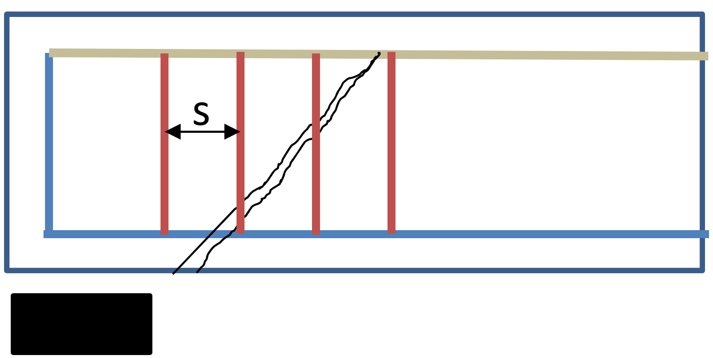
DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

$$s_{l,max} = 0{,}75d(1 + ctg\alpha) =$$

Choosing **s = 150 mm**

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



$$V_{Rd,s} \geq V_{Ed}$$

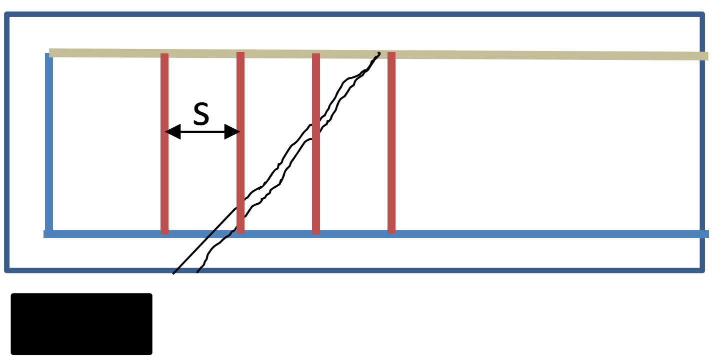
DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

$s_{l,max} = 0{,}75d(1 + ctg\alpha) =$197 mm

Choosing *s = 150 mm*

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



$$V_{Rd,s} \geq V_{Ed}$$

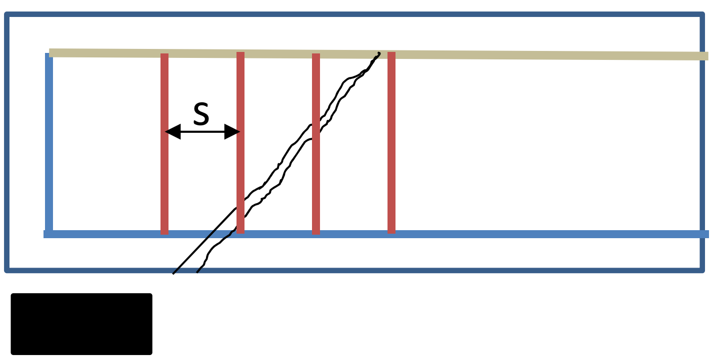
DESIGN FOR SHEAR

CAPACITY OF SHEAR REINFORCEMENT ($V_{Rd,s}$)

$$s_{l,max} = 0{,}75d(1 + ctg\alpha) = 197 \text{ mm}$$

Choosing $s = 150$ *mm*

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot ctg\theta = \mathbf{38.65\ kN}$$



$$V_{Rd,s} \geq V_{Ed}$$

DESIGN FOR SHEAR

DUCTILITY CONDITION FOR SHEAR REINFORCEMENT

$$V_{Rd,s} \leq V_{Rd,max}$$

$$\frac{A_{sw}}{s} \cdot f_{ywd} \leq b_w \cdot \nu_1 \cdot f_{cd} \cdot \frac{1}{1 + ctg^2\theta}$$

For a crack at 45° (*ctgθ = 1*)

→

$$\frac{A_{sw} \cdot f_{ywd}}{b_w \cdot s} \leq 0,5\nu_1 \cdot f_{cd}$$

DESIGN FOR SHEAR

DUCTILITY CONDITION FOR SHEAR REINFORCEMENT

$$\frac{A_{sw} \cdot f_{ywd}}{b_w \cdot s} \leq 0{,}5\nu_1 \cdot f_{cd}$$

? < ?

DESIGN FOR SHEAR

DUCTILITY CONDITION FOR SHEAR REINFORCEMENT

$$\frac{A_{sw} \cdot f_{ywd}}{b_w \cdot s} \leq 0{,}5\nu_1 \cdot f_{cd}$$

$1.09\ N/mm^2 < 3.68\ N/mm^2$

OK!

DESIGN FOR SHEAR

DUCTILITY CONDITION FOR SHEAR REINFORCEMENT

$$\frac{A_{sw} \cdot f_{ywd}}{b_w \cdot s} \leq 0{,}5\nu_1 \cdot f_{cd}$$

1.09 N/mm^2 < 3.68 N/mm^2

OK!

2ND VARIANT!!!

DESIGN OF LONGITUDINAL REINFORCEMENT

AND

SHEAR REINFORCEMENT

USING STIRRUPS AND INCLINED BARS

DESIGN OF LONGITUDINAL REINFORCEMENT

2ND VARIANT

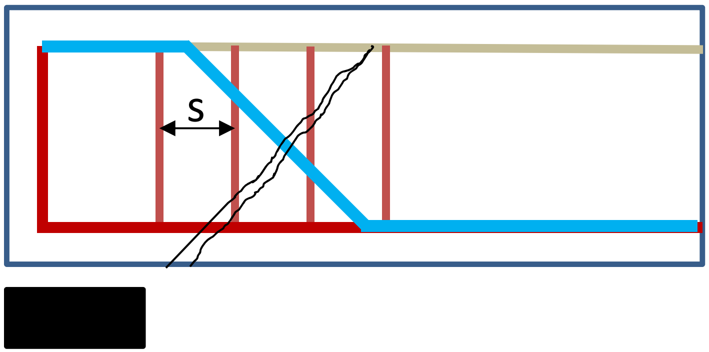
DESIGN OF LONGITUDINAL REINFORCEMENT

$$A_s = \omega_s b d \frac{f_{cd}}{f_{yd}}$$ $= 367\ mm^2$

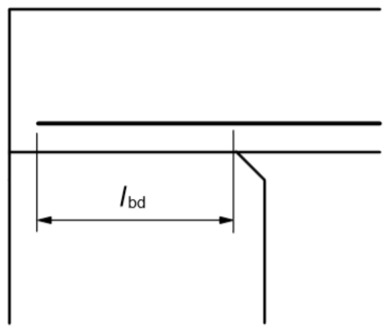
$$\Rightarrow 3\ \phi\ 14 =$$ $462\ mm^2$

$d_s = c_{nom} + \phi_s/2 =$ 38 mm

$d = h - d_s =$ 262 mm



DESIGN OF ANCHORAGE LENGTH



a) Direct support: Beam supported by wall or column

$$f_{bd} = \quad 2{,}25\ \text{N/mm}^2$$

$$l_{b,rqd} = \frac{\phi \cdot f_{yd}}{4 \cdot f_{bd}} \quad = 676\ \text{mm}$$

$$l_{bd} = \alpha_1\alpha_2\alpha_3\alpha_4\alpha_5 l_{b,rqd} \geq l_{b,min} \quad = 676\ \text{mm}$$

DESIGN FOR SHEAR

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix}[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}]b_w d \\ (v_{min} + k_1\sigma_{cp})b_w d\end{pmatrix}$$

$C_{Rd,c} = 0{,}18/\gamma_c =$ 0.12

$k = 1 + \sqrt{\frac{200}{d}} =$ 1.874

$k \leq 2$ √

$k_1 = 0{,}15$ $A.N.$

$\sigma_{cp} = N_{Ed}/A_c < 0{,}2 f_{cd}$ =0

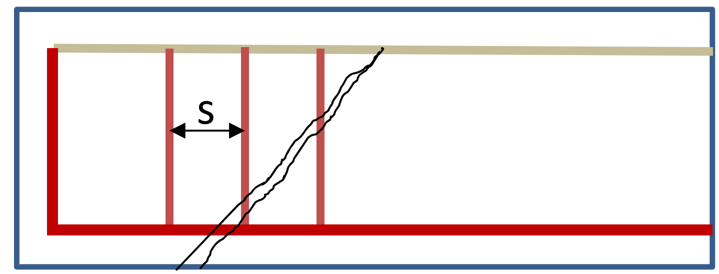
DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix}\left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \\ (v_{min} + k_1\sigma_{cp})b_w d\end{pmatrix}$$

A_{sl} = 2ϕ14 = 308 mm²

$\rho_l = \frac{A_{sl}}{b_w d} =$ 0.0078

$\rho_l \leq 0.02$ √



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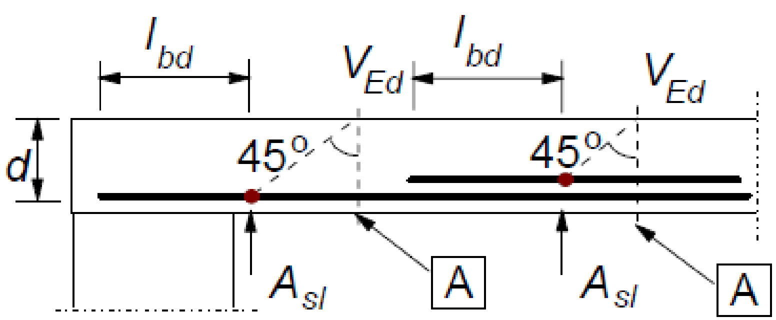
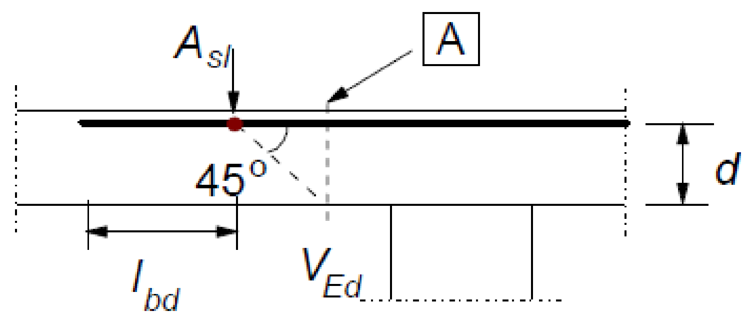




A - section considered

DESIGN FOR SHEAR

$$V_{Rd,c} = \max\begin{pmatrix}\left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \\ (v_{min} + k_1\sigma_{cp})b_w d\end{pmatrix}$$

$v_{min} = 0{,}035k^{3/2} \cdot f_{ck}^{1/2}$= 0.401

$V_{Rd,c} =$ 22.11 kN

DESIGN FOR SHEAR

$$V_{Rd,c} = 22.11\ kN \quad < \quad V_{Ed} = 35\ kN$$

→ SHEAR REINFORCEMENT IS REQUIRED

$$V_{Rd} = \min(V_{Rd,s}; V_{Rd,max})$$

DESIGN FOR SHEAR

CAPACITY OF THE COMPRESSED (CONCRETE) CHORD ($V_{Rd,max}$)

$$V_{Rd,max} = \alpha_{cw} b_w \cdot z \cdot \nu_1 \cdot f_{cd} \frac{(ctg\theta + ctg\alpha)}{1 + ctg^2\theta}$$

α_{cw} - is a coefficient taking account of the state of the stress in the compression chord

$\alpha_{cw} = 1$ for RC elements

$\alpha_{cw} > 1$ for PC elements

ν_1 - is a strength reduction factor for concrete cracked in shear

$\nu_1 = \nu = 0,6\left(1 - \frac{f_{ck}}{250}\right) =$ 0.552 (A.N.)

$z \approx 0.9d =$ 236 mm

DESIGN FOR SHEAR

CAPACITY OF THE COMPRESSED (CONCRETE) CHORD ($V_{Rd,max}$)

$$V_{Rd,max} = \alpha_{cw} b_w \cdot z \cdot \nu_1 \cdot f_{cd} \frac{(ctg\theta + ctg\alpha)}{1 + ctg^2\theta}$$

Choosing

θ = 45° α = 45°

$V_{Rd,max} =$ 260.32 kN

DESIGN FOR SHEAR

SHEAR CAPACITY OF ELEMENTS REINFORCED WITH INCLINED BARS

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd}(ctg\theta + ctg\alpha)sin\alpha$$

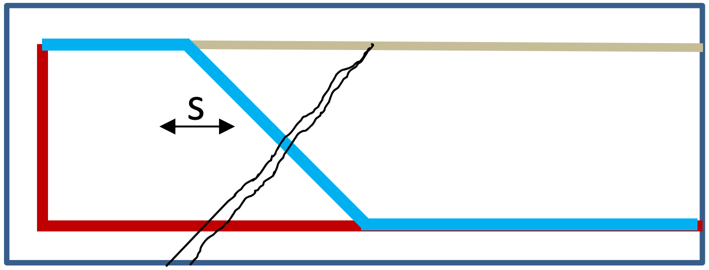
Choosing

A_{sw} - aria of inclined bar

s - distance between inclined bars

θ = 45°

α = 45°



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$A_{sw} = 1\phi 14 = 154\ mm^2$

DESIGN FOR SHEAR

SHEAR CAPACITY OF ELEMENTS REINFORCED WITH INCLINED BARS

Maximum distance between inclined bars

$$s_{b,max} = 0{,}6d(1 + ctg\alpha) =$$ 314 mm

Choosing ***s = 300 mm***

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd}(ctg\theta + ctg\alpha)sin\alpha =$$ **74.4 kN**

DESIGN FOR SHEAR

SHEAR CAPACITY OF ELEMENTS REINFORCED WITH INCLINED BARS

$$V_{Rd,s} \leq V_{Rd,max}$$ √

For an inclined bar at 45° and compressed chord of 45° :

$$\frac{A_{sw} \cdot f_{ywd}}{b_w \cdot s} \leq 0{,}5 \frac{\nu_1 \cdot f_{cd}}{sin\alpha}$$

DESIGN FOR SHEAR

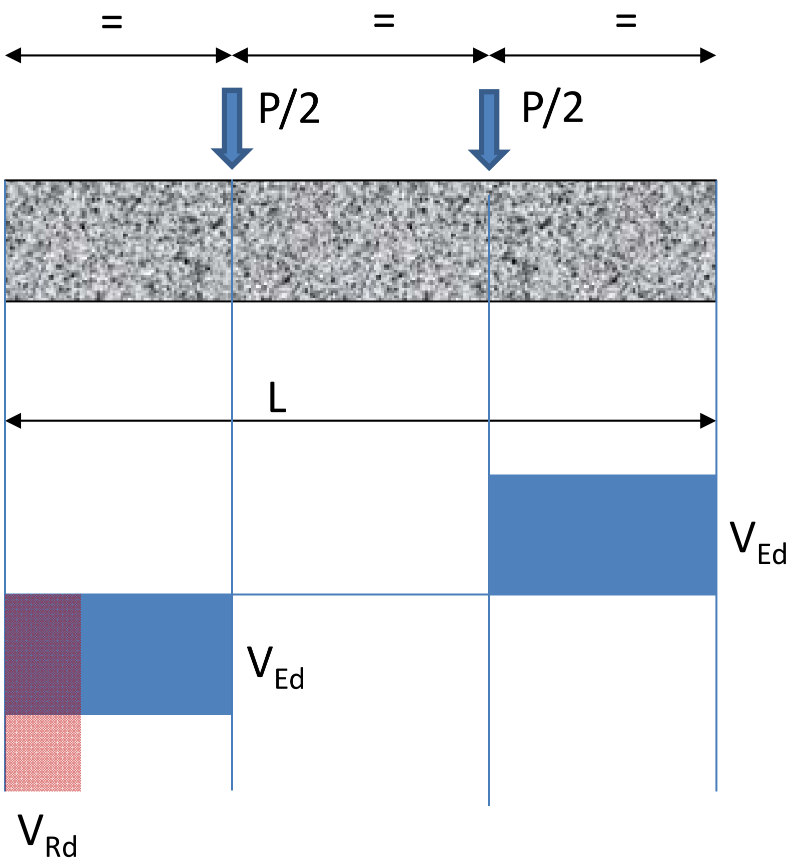
SHEAR CAPACITY OF ELEMENTS REINFORCED WITH INCLINED BARS

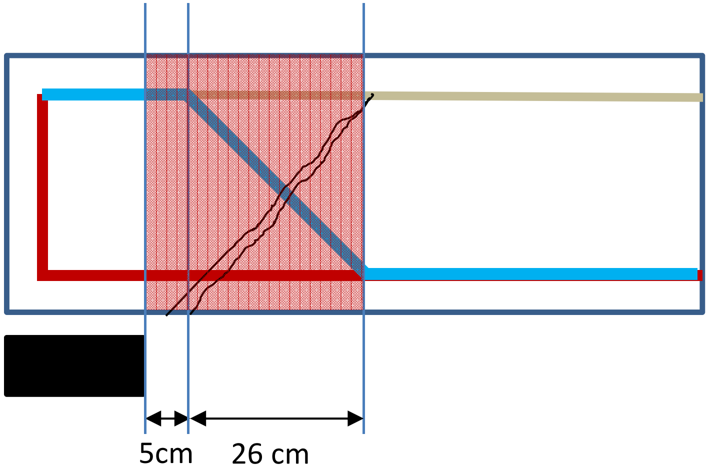
$$\frac{A_{sw} \cdot f_{ywd}}{b_w \cdot s} \leq 0,5 \frac{\nu_1 \cdot f_{cd}}{sin\alpha}$$ √

1.49 N/mm² < 5.20 N/mm² √

DESIGN FOR SHEAR

SHEAR CAPACITY ALONG THE INCLINED BAR ($V_{Rd,s}$)





DESIGN FOR SHEAR

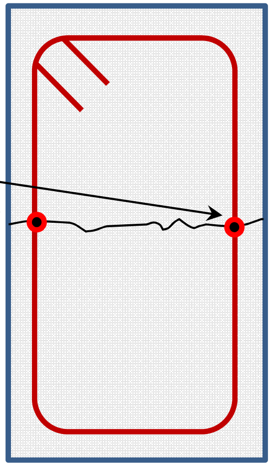
SHEAR CAPACITY AFTER THE INCLINED BAR ($V_{Rd,s}$)

Se impune diametrul (ϕ) etrierului

$\Rightarrow$ $A_{sw}=nA_s$
(n= nr ramuri!!!)

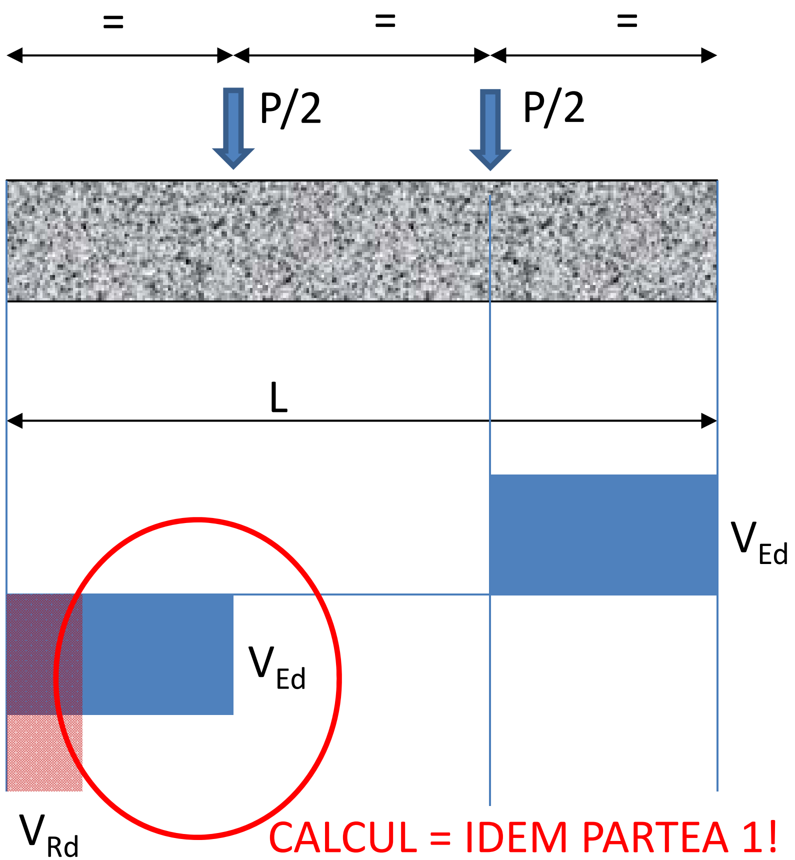
$\Rightarrow$ ***s*** *(pasul)*

$$A_{sw} = 2\phi 6 =$$ 56.5 mm^2



DESIGN FOR SHEAR

SHEAR CAPACITY AFTER THE INCLINED BAR ($V_{Rd,s}$)

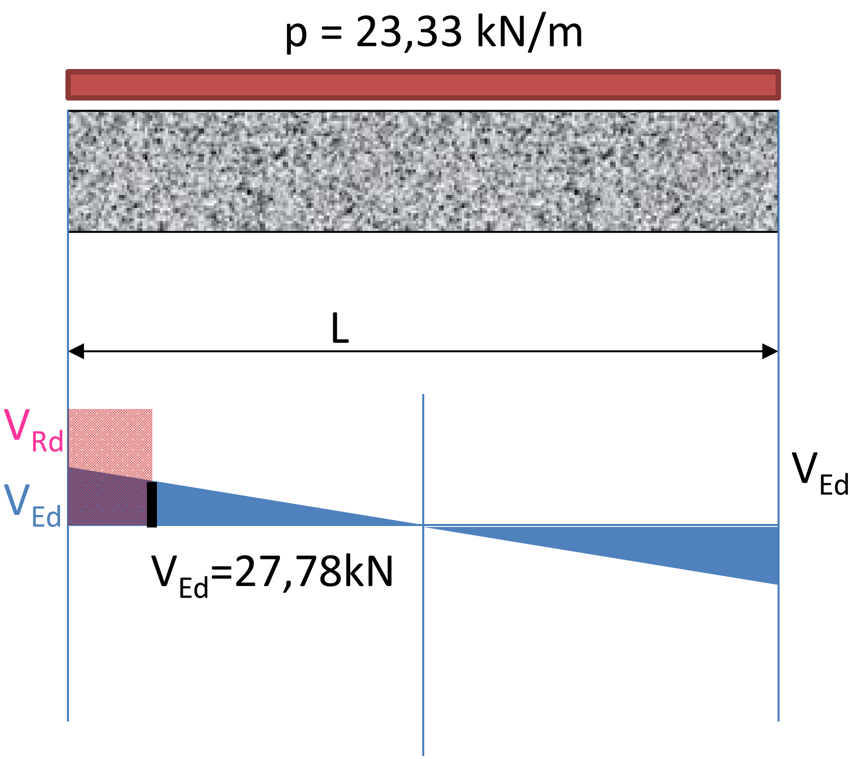


CALCUL = IDEM PARTEA 1!

V_{Ed} = IDEM

DESIGN FOR SHEAR

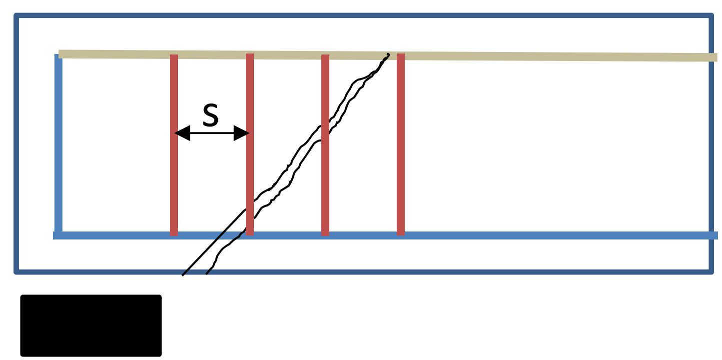
SITUATION WITH DISTRIBUTED LOADS



$$s_{l,max} = 0{,}75d(1 + ctg\alpha) = \text{197 mm}$$

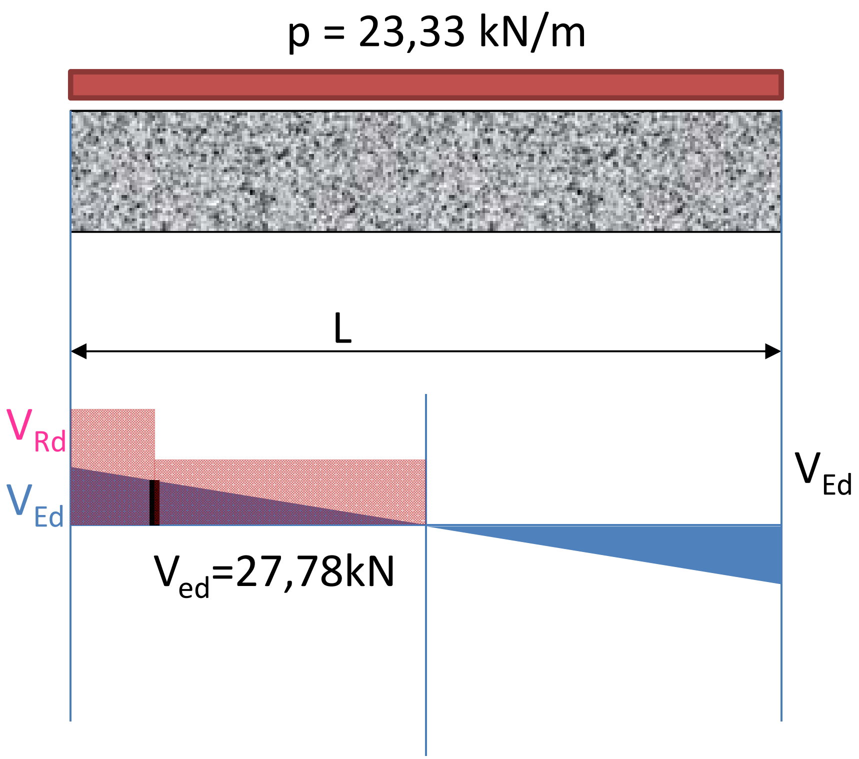
Imposing ***ϕ 6 / 19 cm***

$$\boldsymbol{V_{Rd,s}} = ???$$



DESIGN FOR SHEAR

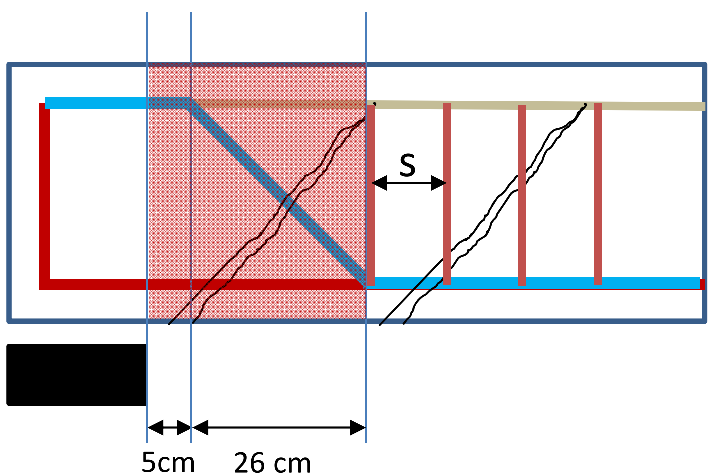
SITUATION WITH DISTRIBUTED LOADS



$$s_{l,max} = 0{,}75d(1 + ctg\alpha) =$$ 197 mm

Imposing **ϕ 6 / 19 cm**

$$\boldsymbol{V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot ctg\theta = 30{,}53kN}$$



THANK YOU FOR YOUR ATTENTIONS!