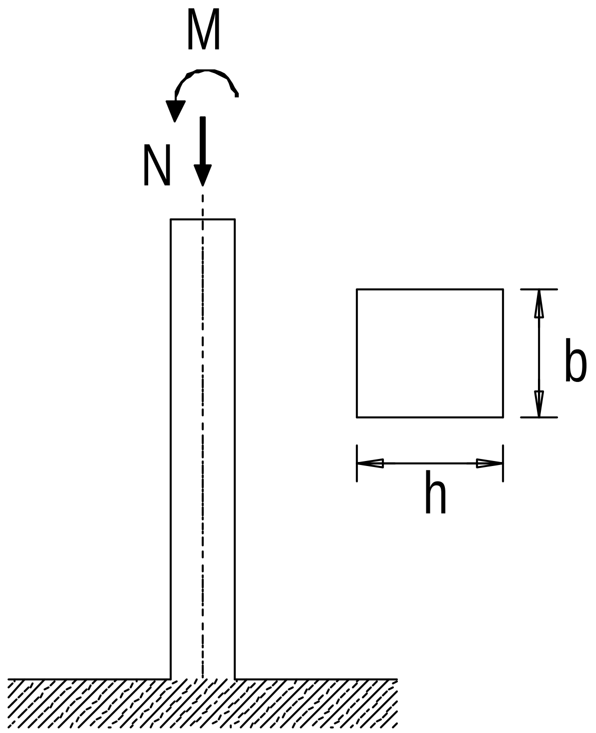
Pos.: Concrete column with bending moment without buckling:

to EN 1992-1-1:2004



Section properties:

Column width b =	25,00 cm
Column thick h =	75,00 cm
Assumed bar size d_{s1} =	2,50 cm
nom_c =	3,50 cm

Design values:

N_{Ed} =	-1980 kN
M_{Ed} =	563 kNm

Materials and characteristic strengths:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
a_{cc} =			1,00
f_{cd} =	$\frac{f_{ck} * a_{cc}}{1,5}$	=	20,00 N/mm²
f_{yd} =	$\frac{f_{yk}}{1,15}$	=	434,78 N/mm²

Analysis:

d_1 =	nom_c + d_{s1}	=	6,00 cm
d_1 / h		=	0,080 ~ 0,1

$\nu_{Ed} = \dfrac{N_{Ed} * 10}{b * h * f_{cd}}$ = -0,528

$\mu_{Ed} = \dfrac{abs(M_{Ed}) * 10^3}{b * h^2 * f_{cd}}$ = 0,200

From the interactive diagram

ω_{tot} = 0,33

$A_{s,tot} = \omega_{tot} * b * h * \dfrac{f_{cd}}{f_{yd}}$ = 28,46 cm²

A_{s1} = $A_{s,tot}$ /2 = 14,23 cm²

A_{s2} = $A_{s,tot}$ /2 = 14,23 cm²

Provide five 20 mm and five 20 mm bars

Longitudinal bars should have a diameter of not less than Æ $d_{s,min}$. The recommended value is:

$d_{s,min}$ = 8,00 mm

d_{s1} =	SEL("reinf/As"; ds; d_s ³ $d_{s,min}$)	=	20 mm
$A_{s,sel1}$ =	SEL("reinf/As"; Name; d_s=d_{s1};As ³ A_{s1})	=	5 Æ 20
prov_A_{s1} =	TAB("reinf/As"; As; Name=$A_{s,sel1}$)	=	15,71 cm²
d_{s2} =	SEL("reinf/As"; ds;)	=	20 mm
$A_{s,sel2}$ =	SEL("reinf/As"; Name; d_s=d_{s2}; As ³ A_{s1})	=	5 Æ 20
prov_A_{s2} =	TAB("reinf/As"; As; Name=$A_{s,sel2}$)	=	15,71 cm²
prov_A_s=	prov_A_{s1} + prov_A_{s2}	=	31,42 cm²

$A_{s,tot}$ / prov_A_s = **0,91 < 1**

constuctive transverse reinforcement

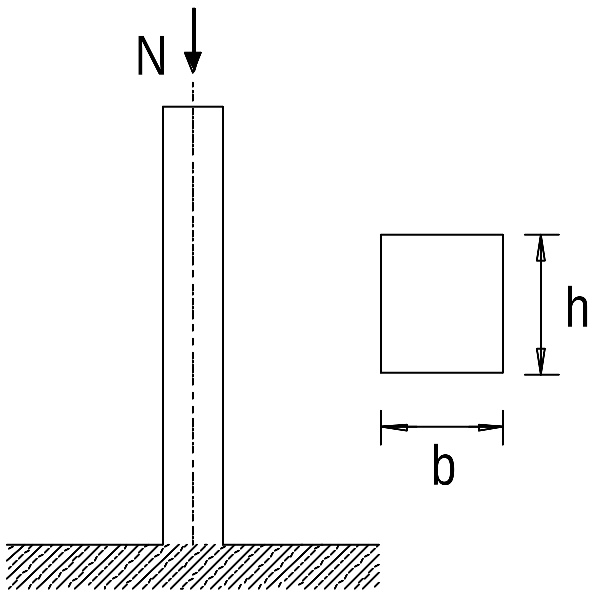
min_d_{sl} =	MIN(d_{s1} ;d_{s2})	=	20,00 mm
min_d_{sw}=			6,00 mm
min_d_{sw}=	0,25*min_d_{sl}	=	5,00 mm
min_d_{sw}=			5,00 mm for mesh reinforceme

The spacing of the transverse reinforcement along the column should not exceed max_s_w

max_s_w=	MIN(20*min_dsl ; 400; MIN(b;h)*10)*10^{-1}	=	25 cm
d_{sw} =	SEL("reinf/AsArea"; ds;)	=	10,00 mm
a_s =	SEL("reinf/AsArea";Name;d_s=d_{sw};e £max_s_w)	=	Æ 10 / e = 20
prov_a_{sw}=	2 * TAB("reinf/AsArea"; as; Name=a_s)	=	7,86 cm²/m

Only closed links are recommended. Hooks are to be staggered.

Pos.: Concrete column without buckling:
to EN 1992-1-1:2004



Section properties:

Column width b =			0,25 m
Column thick h =			0,75 m
Assumed bar size d_{s1} =			2,50 cm
nom_c =			3,50 cm
Concrete cross-sectional area A_c =	b*h	=	0,188 m²

Design forces:

N_{Ed} =	-2800 kN

Materials and characteristic strengths:

Concrete =	SEL("concrete/EC"; Name;)	=	C20/25
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	20,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
$f_{tk,cal}$ =			525,00 N/mm²
E_s =			200000 N/mm²
g_s =			1,15
a cc =			1,00
f_{cd} =	$\frac{f_{ck} * a_{cc}}{1,5}$	=	13,33 N/mm²
$f_{td,cal}$ =	$\frac{f_{tk,cal}}{g_s}$	=	456,52 N/mm²

Analysis:

Value of stress s_{sd} of the bar with $e_s = e_{c2}$

$e_s =$ -2,00 $*10^{-3}$

$$e_{yd} = \frac{f_{yd}}{E_s} = 2{,}174*10^{-3}$$

$e_{su} =$ $25{,}00*10^{-3}$

$$s_{sd} = f_{yd} + \frac{e_s - e_{yd}}{e_{su} - e_{yd}} * (f_{td,cal} - f_{yd}) = 430{,}80 \text{ N/mm}^2$$

$$A_{s,tot} = \frac{N_{Ed} * 10^{-3} + A_c * f_{cd}}{s_{sd}} * 10^4 * (-1) = 6{,}82 \text{ cm}^2$$

$$A_{smin} = \text{MAX}\left(\frac{0{,}1 * \text{abs}(N_{Ed}) * 10^{-3}}{f_{yd}} * 10^4 ;0{,}002 * A_c *10^4\right) = 6{,}44 \text{ cm}^2$$

$A_{smin} / A_{s,tot}$ = **0,94 £1**

$A_s =$ MAX($A_{s,tot}$;A_{smin}) = **6,82 cm²**

Provide 4 Æ 20 mm bars

Longitudinal bars should have a diameter of not less than Æ $d_{sl,min}$. The recommended value is:

$d_{sl,min} =$ 8,00 mm

$d_s =$ SEL("reinf/As"; ds; d_s ³ $d_{sl,min}$) = 20 mm

$A_{s,sel} =$ SEL("reinf/As"; Name; d_s=d_s; As ³ A_s) = 4 Æ 20

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 12,57 cm²

A_s / prov_A_s = **0,54 < 1**

min_d_{sw}= 6,00 mm

min_d_{sw}= 0,25 * d_s = 5,00 mm

min_d_{sw}= 5,00 mm f. mesh reinforcement

The spacing of the transverse reinforcement along the column should not exceed max_s_w

max_s_w= MIN(20*$d_{sl,min}$; 400; (MIN(b;h))*10^3)*10^{-1} = 16 cm

d_{sw} = SEL("reinf/AsArea"; ds;) = 10,00 mm

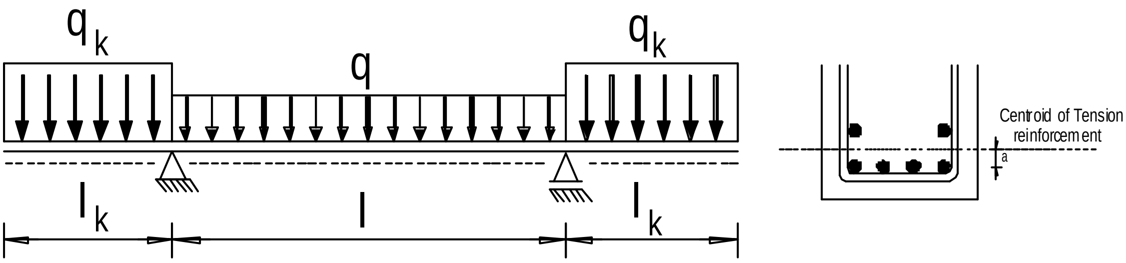
a_s = SEL("reinf/AsArea";Name;d_s=d_{sw};e £max_s_w) = Æ 10 / e = 16

prov_a_{sw}= 2 * TAB("reinf/AsArea"; as; Name=a_s) = 9,82 cm²/m

Only closed links are recommended. The hooks are to be curtailed.

Pos.: Single-span with cantilever^s:

to EN 1992-1-1:2004



Section properties:

Span l = 2,80 m
Length of cantilever l_{cl} = 0,80 m
Length of cantilever l_{cr} = 1,00 m
Width b = 0,24 m
Depth h = 0,55 m
Centroid of tension reinforcement a = 0,02 m
Assumed bar size d_{s1} = 0,025 m
nom_c = 0,035 m

Materials and characteristic strengths:

Concrete = SEL("concrete/EC"; Name;$f_{ck} \leq$ 50) = C30/37
Steel = SEL("reinf/Steel"; Name;) = 500 S
f_{ck} = TAB("concrete/EC"; fck; Name=Concrete) = 30,00 N/mm²
f_{yk} = TAB("reinf/Steel"; βs; Name=Steel) = 500 MN/m²
f_{yd}= f_{yk} / 1,15 = 434,78 N/mm²
α_{cc} = 1,00

$$f_{cd} = \frac{f_{ck} * \alpha_{cc}}{1,5} = 20,00 \text{ N/mm}^2$$

Partial safety factors:

γ_G = 1,35
γ_Q = 1,50

Actions on span:

from dead load: b * h * 25 = 3,30 kN/m
from nib: b * 1,35 * 25 = 8,10 kN/m
from Pos. 201: 48,50 kN/m

max q_{gf} = 59,90 kN/m

live load from Pos1: 20,00 kN/m
live load from Pos2: 74,50 kN/m

max q_{qf} = 94,50 kN/m

Actions on left cantilever:

from dead load:	b * h * 25	=	3,30 kN/m
from nib:	b * 1,35 * 25	=	8,10 kN/m
from Pos. 201:			48,50 kN/m
		max q_{gcl} =	**59,90 kN/m**
live load from Pos1:			40,00 kN/m
live load from Pos2:			70,50 kN/m
		max q_{qcl} =	**110,50 kN/m**

Actions on right cantilever:

from dead load:	b * h * 25	=	3,30 kN/m
from nib:	b * 1,35 * 25	=	8,10 kN/m
from Pos. 201:			48,50 kN/m
		max q_{gcr} =	**59,90 kN/m**
live load from Pos1:			40,00 kN/m
live load from Pos2:			70,50 kN/m
		max q_{qcr} =	**110,50 kN/m**

Design values and analysis:

$$M_{gcl} = \frac{-q_{gcl} * l_{cl}^2}{2} = -19,17 \text{ kNm}$$

$$M_{qcl} = \frac{-q_{qcl} * l_{cl}^2}{2} = -35,36 \text{ kNm}$$

$$M_{gcr} = \frac{-q_{gcr} * l_{cr}^2}{2} = -29,95 \text{ kNm}$$

$$M_{qcr} = \frac{-q_{qcr} * l_{cr}^2}{2} = -55,25 \text{ kNm}$$

$$M_{gfield} = \frac{q_{gf} * l^2}{8} = 58,70 \text{ kNm}$$

$$M_{qfield} = \frac{q_{qf} * l^2}{8} = 92,61 \text{ kNm}$$

Design values and analysis left cantilever:

M_{Edcl} =	$g_G * M_{gcl} + g_Q * M_{qcl}$	=	-78,92 kNm
d =	h - nom_c - a - d_{s1} / 2	=	0,482 m
z_{s1} =	d - h / 2	=	0,21 m
$M_{Edc,s}$ =	ABS(M_{Edcl})	=	78,92 kNm
$m_{Edcl,s}$ =	$\frac{M_{Edc,s} * 10^{-3}}{b * d^{2} * f_{cd}}$	=	0,0708
w =	TAB("reinf/Ecmy"; w; m=$m_{Edcl,s}$)	=	0,075
req_A_s=	$\frac{w * d * b * f_{cd}}{f_{yd}} * 10^{4}$	=	3,99 cm²

Provide three 16 mm bars

Bar size d_s =	SEL("reinf/As"; ds;)	=	16 mm
$A_{s,sel}$ =	SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s)	=	3 Æ 16
prov_A_s =	TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	6,03 cm²

req_A_s / prov_A_s = **0,66 < 1**

Design values and analysis right cantilever:

M_{Edcr} =	$g_G * M_{gcr} + g_Q * M_{qcr}$	=	-123,31 kNm
d =	h - nom_c - a - d_{s1} / 2	=	0,482 m
z_{s1} =	d - h / 2	=	0,21 m
$M_{Edcr,s}$ =	ABS(M_{Edcr})	=	123,31 kNm
$m_{Edcr,s}$ =	$\frac{M_{Edcr,s} * 10^{-3}}{b * d^{2} * f_{cd}}$	=	0,111
w =	TAB("reinf/Ecmy"; w; m=$m_{Edcr,s}$)	=	0,118
req_A_s=	$\frac{w * d * b * f_{cd}}{f_{yd}} * 10^{4}$	=	6,28 cm²

Provide five 14 mm bars

Bar size d_s =	SEL("reinf/As"; ds;)	=	14 mm
$A_{s,sel}$ =	SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s)	=	5 Æ 14
prov_A_s =	TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	7,70 cm²

req_A_s / prov_A_s = **0,82 < 1**

Design values and analysis span:

$M_{Edf,min}$ = $M_{gfield} + \dfrac{M_{Edcl} + M_{Edcr}}{2}$ = -42,41 kNm

d = h - nom_c - a - d_{s1} / 2 = 0,482 m

z_{s1} = d - h / 2 = 0,21 m

$M_{Edf,min,s}$ = ABS($M_{Edf,min}$) = 42,41 kNm

$m_{Edf,min,s}$ = $\dfrac{M_{Edf,min,s} * 10^{-3}}{b * d^{2} * f_{cd}}$ = 0,038

w = TAB("reinf/Ecmy"; w; m=$m_{Edf,min,s}$) = 0,039

req_A_s= $\dfrac{w * d * b * f_{cd}}{f_{yd}} * 10^{4}$ = 2,08 cm²

Provide two 14 mm bars

Bar size d_s = SEL("reinf/As"; ds;) = 14 mm

$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s) = 2 Æ 14

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 3,08 cm²

req_A_s / prov_A_s = **0,68 < 1**

$M_{Edf,max}$ = $g_G * M_{gfield} + g_Q * M_{qfield} + \dfrac{M_{gcl} + M_{gcr}}{2}$ = 193,60 kNm

d = h - nom_c - a - d_{s1} / 2 = 0,482 m

z_{s1} = d - h / 2 = 0,21 m

$M_{Edf,max,s}$ = ABS($M_{Edf,max}$) = 193,60 kNm

$m_{Edf,max,s}$ = $\dfrac{M_{Edf,max,s} * 10^{-3}}{b * d^{2} * f_{cd}}$ = 0,174

w = TAB("reinf/Ecmy"; w; m=$m_{Edf,max,s}$) = 0,193

req_A_s= $\dfrac{w * d * b * f_{cd}}{f_{yd}} * 10^{4}$ = 10,27 cm²

Provide four 20 mm bars

Bar size d_s = SEL("reinf/As"; ds;) = 20 mm

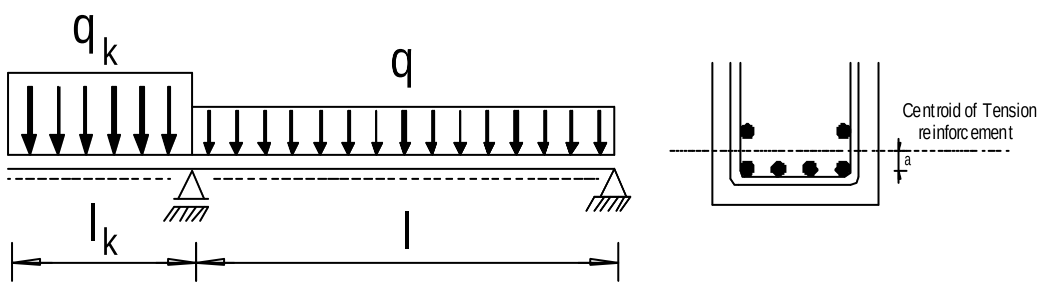
$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s) = 4 Æ 20

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 12,57 cm²

req_A_s / prov_A_s = **0,82 < 1**

Pos.: Single-span with left cantilever:

to EN 1992-1-1:2004



Section properties:

Span l =	2,80 m
Length of cantilever l_c =	1,00 m
Width b =	0,25 m
Depth h =	0,55 m
Centroid of tension reinforcement a =	0,02 m
Assumed bar size d_{s1} =	0,025 m
nom_c =	0,035 m

Materials and characteristic strengths:

Concrete =	SEL("concrete/EC"; Name;f_{ck} £ 50)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a cc =			1,00
f_{cd} =	$\frac{f_{ck} * a_{cc}}{1,5}$	=	20,00 N/mm²

Partial safety factors:

g_G =	1,35
g_Q =	1,50

Actions on span:

from dead load:	b * h * 25	=	3,44 kN/m
from nib:	b * 1,35 * 25	=	8,44 kN/m
from Pos. 201:			48,50 kN/m
		max q_g =	**60,38 kN/m**

live load from Pos1:	20,00 kN/m
live load from Pos2:	74,50 kN/m
max q_p =	**94,50 kN/m**

Actions on cantilever:

from dead load:	b * h * 25	=	3,44 kN/m
from nib:	b * 1,35 * 25	=	8,44 kN/m
from Pos. 201:			48,50 kN/m
		max q_{gc} =	**60,38 kN/m**

live load from Pos1:			40,00 kN/m
live load from Pos2:			70,50 kN/m
		max q_{pc} =	**110,50 kN/m**

Design values and analysis:

$$M_{gcl} = \frac{-q_{gc} * l_c^{\ 2}}{2} = -30{,}19 \text{ kNm}$$

$$M_{pcl} = \frac{-q_{pc} * l_c^{\ 2}}{2} = -55{,}25 \text{ kNm}$$

$$M_{gfield} = \frac{q_g * l^2}{8} = 59{,}17 \text{ kNm}$$

$$M_{pfield} = \frac{q_p * l^2}{8} = 92{,}61 \text{ kNm}$$

Design values and analysis cantilever:

M_{Edc} = $g_G * M_{gcl} + g_Q * M_{pcl}$ = -123,63 kNm

d = h - nom_c - a - d_{s1} / 2 = 0,482 m

z_{s1} = d - h / 2 = 0,21 m

$M_{Edc,s}$ = ABS(M_{Edc}) = 123,63 kNm

$$m_{Edc,s} = \frac{M_{Edc,s} * 10^{-3}}{b * d^2 * f_{cd}} = 0{,}1064$$

w = TAB("reinf/Ecmy"; w; m=$m_{Edc,s}$) = 0,113

$$req\_A_s = \frac{w * d * b * f_{cd}}{f_{yd}} * 10^4 = 6{,}26 \text{ cm}^2$$

Provide four 25 mm bars

Bar size d_s = SEL("reinf/As"; ds;) = 25 mm

$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s) = 4 Æ 25

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 19,63 cm²

req_A_s / prov_A_s = **0,32 < 1**

Design values and analysis span:

$M_{Edf,min}$ =	$M_{gfield} + \frac{M_{Edc}}{2}$	=	-2,65 kNm
d =	h - nom_c - a - d_{s1} / 2	=	0,482 m
z_{s1} =	d - h / 2	=	0,21 m
$M_{Edf,min,s}$ =	ABS($M_{Edf,min}$)	=	2,65 kNm
$m_{Edf,min,s}$ =	$\frac{M_{Edf,min,s} * 10^{-3}}{b * d^2 * f_{cd}}$	=	0,0023
w =	TAB("reinf/Ecmy"; w; m=$m_{Edf,min,s}$)	=	0,002
req_A_s=	$\frac{w * d * b * f_{cd}}{f_{yd}} * 10^4$	=	0,11 cm²

Provide two 16 mm bars

Bar size d_s =	SEL("reinf/As"; ds;)	=	16 mm
$A_{s,sel}$ =	SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s)	=	2 Æ 16
prov_A_s =	TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	4,02 cm²

req_A_s / prov_A_s = **0,03 < 1**

$M_{Edf,max}$ =	$g_G * M_{gfield} + g_Q * M_{pfield} + \frac{M_{gcl}}{2}$	=	203,70 kNm
d =	h - nom_c - a - d_{s1} / 2	=	0,482 m
z_{s1} =	d - h / 2	=	0,21 m
$M_{Edf,max,s}$ =	ABS($M_{Edf,max}$)	=	203,70 kNm
$m_{Edf,max,s}$ =	$\frac{M_{Edf,max,s} * 10^{-3}}{b * d^2 * f_{cd}}$	=	0,175
w =	TAB("reinf/Ecmy"; w; m=$m_{Edf,max,s}$)	=	0,194
req_A_s=	$\frac{w * d * b * f_{cd}}{f_{yd}} * 10^4$	=	10,75 cm²

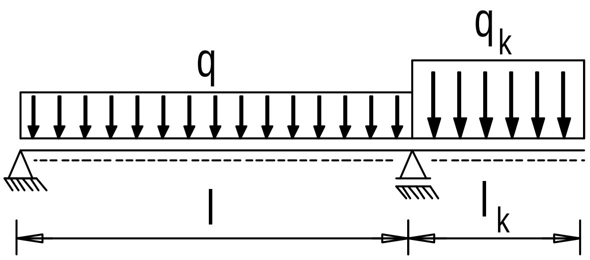
Provide four 20 mm bars

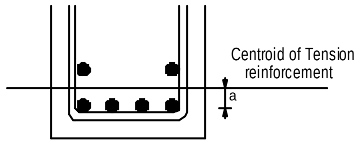
Bar size d_s =	SEL("reinf/As"; ds;)	=	20 mm
$A_{s,sel}$ =	SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s)	=	4 Æ 20
prov_A_s =	TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	12,57 cm²

req_A_s / prov_A_s = **0,86 < 1**

Pos.: Single-span with right cantilever:

to EN 1992-1-1:2004





Section properties:

Span l =	2,80 m
Length of cantilever l_c =	1,60 m
Width b =	0,30 m
Depth h =	0,55 m
Centroid of tension reinforcement a =	0,02 m
Assumed bar size d_{s1} =	0,025 m
nom_c =	0,035 m

Materials and characteristic strengths:

Concrete =	SEL("concrete/EC"; Name; f_{ck} £ 50)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a_{cc} =			1,00
f_{cd} =	$\frac{f_{ck} * a_{cc}}{1,5}$	=	20,00 N/mm²

Partial safety factors:

g_G =	1,35
g_Q =	1,50

Actions on span:

from dead load:	b * h * 25	=	4,13 kN/m
from nib:	b * 1,35 * 25	=	10,13 kN/m
from Pos. 201:			48,50 kN/m
		max q_g =	**62,76 kN/m**

live load from Pos1:	20,00 kN/m
live load from Pos2:	74,50 kN/m
max q_p =	**94,50 kN/m**

Actions on cantilever:

from dead load:	b * h * 25	=	4,13 kN/m
from nib:	b * 1,35 * 25	=	10,13 kN/m
from Pos. 201:			48,50 kN/m
		max q_{gc} =	**62,76 kN/m**

live load from Pos1:			40,00 kN/m
live load from Pos2:			70,50 kN/m
		max q_{pc} =	**110,50 kN/m**

Design values and analysis:

$$M_{gcr} = \frac{-q_{gc} * l_c^{2}}{2} = -80,33 \text{ kNm}$$

$$M_{pcr} = \frac{-q_{pc} * l_c^{2}}{2} = -141,44 \text{ kNm}$$

$$M_{gfield} = \frac{q_g * l^{2}}{8} = 61,50 \text{ kNm}$$

$$M_{pfield} = \frac{q_p * l^{2}}{8} = 92,61 \text{ kNm}$$

Design values and analysis cantilever:

$$M_{Edc} = g_G * M_{gcr} + g_Q * M_{pcr} = -320,61 \text{ kNm}$$

$$d = h - nom\_c - a - d_{s1} / 2 = 0,482 \text{ m}$$

$$z_{s1} = d - h / 2 = 0,21 \text{ m}$$

$$M_{Edc,s} = ABS(M_{Edc}) = 320,61 \text{ kNm}$$

$$m_{Edc,s} = \frac{M_{Edc,s} * 10^{-3}}{b * d^{2} * f_{cd}} = 0,230$$

$$w = TAB(\text{"reinf/Ecmy"}; w; m = m_{Edc,s}) = 0,267$$

$$req\_A_s = \frac{w * d * b * 10^{4} * f_{cd}}{f_{yd}} = 17,76 \text{ cm}^2$$

Provide four 25 mm bars

Bar size d_s =	SEL("reinf/As"; ds;)	=	25 mm
$A_{s,sel}$ =	SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s)	=	4 Æ 25
prov_A_s =	TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	19,63 cm²

req_A_s / prov_A_s = **0,90 < 1**

Design values and analysis span:

$M_{Edf,min} = \quad M_{gfield} + \frac{M_{Edc}}{2} \quad = \quad -98{,}81$ kNm

$d = \quad h - nom\_c - a - d_{s1} / 2 \quad = \quad 0{,}482$ m

$z_{s1} = \quad d - h / 2 \quad = \quad 0{,}21$ m

$M_{Edf,min,s} = \quad ABS(M_{Edf,min}) \quad = \quad 98{,}81$ kNm

$m_{Edf,min,s} = \quad \frac{M_{Edf,min,s} * 10^{-3}}{b * d^{2} * f_{cd}} \quad = \quad 0{,}071$

$w = \quad TAB("reinf/Ecmy"; w; m=m_{Edf,min,s}) \quad = \quad 0{,}075$

$req\_A_s = \quad \frac{w * d * b * 10^{4} * f_{cd}}{f_{yd}} \quad = \quad 4{,}99$ cm²

Provide two 25 mm bars

Bar size $d_s =$ SEL("reinf/As"; ds;) = 25 mm

$A_{s,sel} =$ SEL("reinf/As"; Name; $d_s = d_s$; As ³ req_A_s) = 2 Æ 25

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 9,82 cm²

req_A_s / prov_A_s = **0,51 < 1**

$M_{Edf,max} = \quad g_G * M_{gfield} + g_Q * M_{pfield} + \frac{M_{gcr}}{2} \quad = \quad 181{,}78$ kNm

$d = \quad h - nom\_c - a - d_{s1} / 2 \quad = \quad 0{,}482$ m

$z_{s1} = \quad d - h / 2 \quad = \quad 0{,}21$ m

$M_{Edf,max,s} = \quad ABS(M_{Edf,max}) \quad = \quad 181{,}78$ kNm

$m_{Edf,max,s} = \quad \frac{M_{Edf,max,s} * 10^{-3}}{b * d^{2} * f_{cd}} \quad = \quad 0{,}130$

$w = \quad TAB("reinf/Ecmy"; w; m=m_{Edf,max,s}) \quad = \quad 0{,}140$

$req\_A_s = \quad \frac{w * d * b * 10^{4} * f_{cd}}{f_{yd}} \quad = \quad 9{,}31$ cm²

Provide four 20 mm bars

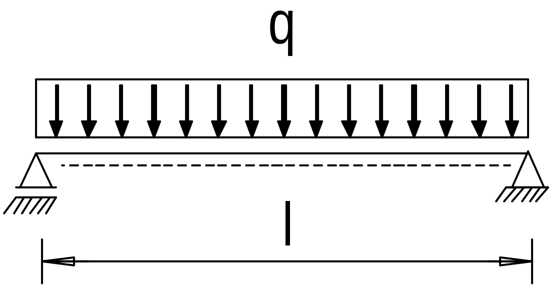
Bar size $d_s =$ SEL("reinf/As"; ds;) = 20 mm

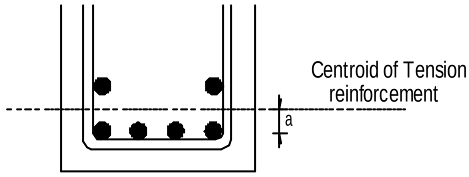
$A_{s,sel} =$ SEL("reinf/As"; Name; $d_s = d_s$; As ³ req_A_s) = 4 Æ 20

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 12,57 cm²

req_A_s / prov_A_s = **0,74 < 1**

Pos.: Single-beam
to EN 1992-1-1:2004





Section properties:

Span L =	2,80 m
Width b =	0,24 m
Depth h =	0,58 m
Centroid of tension reinforcement a =	0,02 m
Assumed bar size d_{s1} =	0,025 m
nom_c =	0,035 m

Materials and characteristic strengths:

Concrete =	SEL("concrete/EC"; Name; $f_{ck} \le 50$)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; βs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
α_{cc} =			1,00
f_{cd} =	$\frac{f_{ck} * \alpha_{cc}}{1,5}$	=	20,00 N/mm²

Partial safety factors:

γ_G =	1,35
γ_Q =	1,50

Actions on span:

from dead load:	b * h * 25	=	3,48 kN/m
from nib:	b * 1,35 * 25	=	8,10 kN/m
from Pos. 201:			48,50 kN/m
		max q_g =	**60,08 kN/m**

live load from Pos1:		20,00 kN/m
live load from Pos2:		37,50 kN/m
	max q_q =	**57,50 kN/m**

Design values and analysis:

M_G =	$\dfrac{q_g * L^2}{8}$	=	58,88 kNm
M_Q =	$\dfrac{q_q * L^2}{8}$	=	56,35 kNm
M_{Ed} =	$g_G * M_G + g_Q * M_Q$	=	164,01 kNm
d =	$h - nom\_c - a - d_{s1} / 2$	=	0,512 m
z_{s1} =	d - h / 2	=	0,22 m
$M_{Ed,s}$ =	ABS(M_{Ed})	=	164,01 kNm
$m_{Ed,s}$ =	$\dfrac{M_{Ed,s} * 10^{-3}}{b * d^2 * f_{cd}}$	=	0,130
w =	TAB("reinf/Ecmy"; w; m=$m_{Ed,s}$)	=	0,140
req_A_s=	$\dfrac{w * d * b * 10^4 * f_{cd}}{f_{yd}}$	=	7,91 cm²

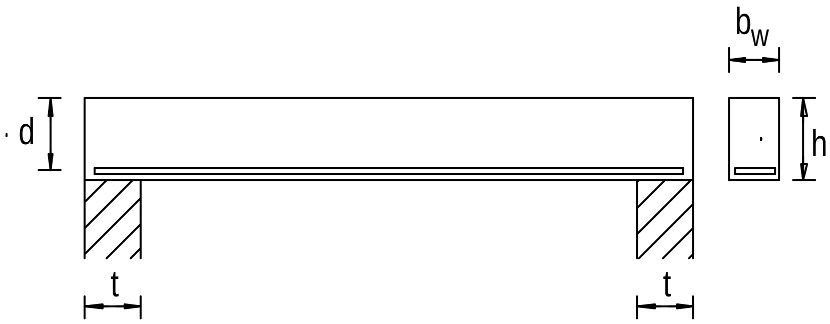
Provide four 16 mm bars

Bar size d_s =	SEL("reinf/As"; ds;)	=	16 mm
$A_{s,sel}$ =	SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s)	=	4 Æ 16
prov_A_s =	TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	8,04 cm²

req_A_s / prov_A_s = **0,98 < 1**

Analysis for shearing without transverse reinforcement:

to EN 1992-1-1:2004



Section properties:

thickness t =		0,30 m
Rib width b_w =		0,30 m
height h =		0,42 m
effective depth of tension reinforcement		
depth d =		0,375 m
Tension reinforcement A_{s1} =		13,23 cm²
length of span l =		3,00 m

Loads:

g =			20,20 kN/m
q =			9,41 kN/m
V_G =	g * l/2	=	30,30 kN
V_Q =	q * l/2	=	14,12 kN

Materials, stresses and partial safety factors:

Concrete =	SEL("concrete/EC"; Name; f_{ck} £ 50)	=	C20/25
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	20,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a **cc** =			1,00
f_{cd} =	$\frac{f_{ck} * a_{cc}}{1,5}$	=	13,33 N/mm²
g_G=	1,35		
g_Q=	1,50		
g_c=	1,50		

Shear Analysis:

max_V =	$V_G * g_G + V_Q * g_Q$	=	62,09 kN
a_i =	MIN(1/2*h ;1/2*t)	=	0,15 m

Effective shearing force:

V_{Ed} =	max_V - (g * g_G + q * g_Q) * (a_i + d)	=	40,36 kN
k =	$\text{MIN}(1+\sqrt{\frac{200}{d * 10^3}} ; 2)$	=	1,73

$\rho_1 =$ $\mathrm{MIN}\left(\dfrac{A_{s1}}{b_w \cdot d \cdot 10^4}\ ;\ 0{,}02\right)$ = 0,012

$k_1 =$ 0,15

$C_{Rd,c} =$ $0{,}18/\gamma_c$ = 0,12

<u>Acceptance:</u>

Concrete compressive stress at the centroidal axis due to axial loading and/or presstressing σ_{cp}=0;

$\sigma_{cp} =$ 0,00 N/mm²

$V_{Rd,c} =$ $\left(C_{Rd,c} * k * \sqrt[3]{100 * \rho_1 * f_{ck}} + k_1 * \sigma_{cp}\right) * b_w * d * 10^3$ = 67,37 kN

$\nu_{min} =$ $0{,}035 * \sqrt{k^3} * \sqrt{f_{ck}}$ = 0,36 N/mm²

$V_{Rd,c,min} =$ $(\nu_{min} + k_1 * \sigma_{cp}) * b_w * d * 10^3$ = 40,50 kN

$V_{Rd,c} =$ $\mathrm{MAX}(V_{Rd,c}\ ;\ V_{Rd,c,min})$ = 67,37 kN

$V_{Ed} / V_{Rd,c}$ = **<u>0,60 ≤ 1</u>**

Shear reinforcement is not necessary, if proof is ≤ 1.

Constructive shearing reinforcement

$\rho_{w,min} =$ $0{,}08 * \dfrac{\sqrt{f_{ck}}}{f_{yk}}$ = $0{,}72 \cdot 10^{-3}$

$s_{t,max} =$ MIN(0,75*d; 0,60) = 0,28 m

α = 90,00°

$a_{sw,min} =$ $\rho_{w,min} * 10^4 * b_w * \mathrm{SIN}(90)$ = 2,16 cm²/m

Provide links of size 8 mm (double-shear)

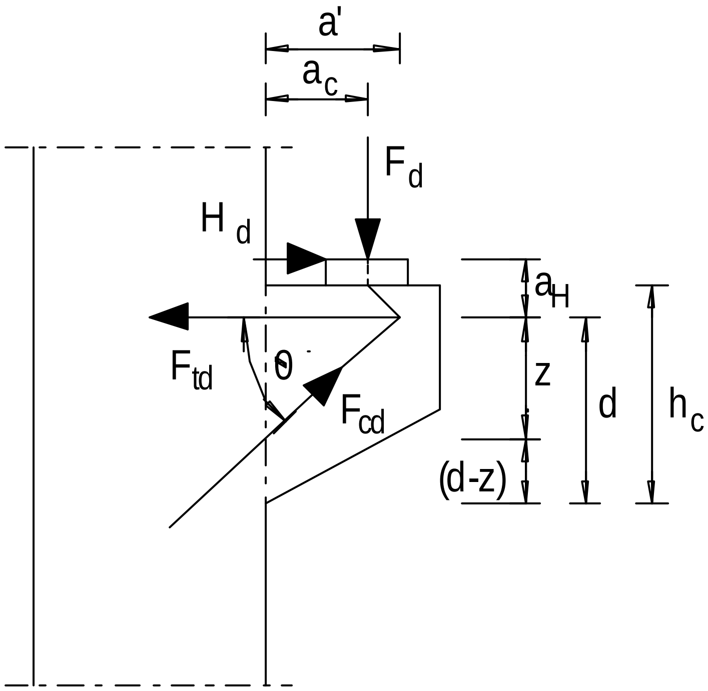
$d_s =$ SEL("reinf/AsArea"; ds;) = 8,00 mm

$a_s =$ SEL("reinf/AsArea";Name;d_s=d_s;$a_s \geq a_{sw,min}/2$) = ∅ 8 / e = 20

prov_a_{sw}= 2 * TAB("reinf/AsArea"; as; Name=a_s) = 5,02 cm²/m

$a_{sw,min}$ / prov_a_{sw} = **<u>0,43 < 1</u>**

Concrete corbel, design with the strut and tie system:
to EN 1992-1-1:2004



Section properties:

Shear span a_c =	20,00 cm
Depth of corbel h_c =	60,00 cm
Thickness of bearing plate h_{pl} =	4,00 cm
Center of reinforcement d_1 =	4,00 cm
Width of corbel b_w =	50,00 cm
dimensions bearing in x direction b_x =	120,00 mm
dimensions bearing in y direction b_y =	350,00 mm

Materials, stresses and partial safety factors:

Concrete =	SEL("concrete/EC"; Name;)	=	C50/60
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	50,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
f_{cd}=	f_{ck} / 1,5	=	33,33 N/mm²
a_{cc} =			1,00

Design values:

F_d =	250,00 kN
H_d =	45,00 kN

Design values and analysis:

Distance a_H =	h_{pl} + d_1	=	8,00 cm
H_d =	MAX(0,2*F_d ; H_d)	=	50,00 kN
d =	h_c - d_1	=	56,00 cm

v = $0,6 * \left(1 - \frac{f_{ck}}{250}\right)$ = 0,48

Safe bearing stress s_{sb} = $0,48 * \left(1 - \frac{f_{ck}}{250}\right) * f_{ck}$ = 19,20 N/mm²

Actual bearing stress s_{ab} = $\frac{F_d * 10^3}{b_x * b_y}$ = 5,95 N/mm²

Check the bearing stress

$\frac{s_{ab}}{s_{sb}}$ = **0,31 £1**

Concrete strut

effective depth d = $h_c - d_1$ = 56,00 cm

Distance a' = $(a_c + 0,2 * a_H)$ = 21,60 cm

Therefore e_1 = $\frac{a'}{d}$ = 0,4

The design stress for the concrete strut

f_{cd} = $v * f_{ck} / 1,5$ = 16,00 N/mm²

Angle of inclination :

$$\frac{F_{Ed}}{f_{cd} * d * b_w} = \left(1 - \frac{a'}{d} * \tan(Q)\right) * \sin(2 * Q)$$

This equation cannot be solved directly for Q but following table, which has been developed directly from ecuation can be used.

Value f_1 = $\frac{F_d * 10^3}{f_{cd} * d * b_w * 10^2}$ = 0,056

Q = TAB("concrete/InclC"; Theta ; e ³ e_1 ; f = f_1) = 39,00 °

z_0 = $(a_c + 0,2 * a_H) * TAN(Q)$ = 17,49 cm

a_c / z_0 = **1,14 < 1 otherwise design as cantilever**

Corbels ($a_c < z_0$) may be designed using strut-and-tie models

Main tension steel

The force in the main tension steel is

F'_{td} = $F_d * \left(\frac{1}{\tan(Q)} + 0,2\right)$ = 358,72 kN

$A_{s,main}$ = $\frac{F'_{td} * 10^3}{f_{yd}} * 10^{-2}$ = 8,25 cm²

Provide two loops H20 mm bars

$d_{s,sel}$ = SEL("reinf/As"; ds;) = 20 mm

$A_{s,sel}$ = SEL("reinf/As"; Name; $d_s = d_{s,sel}$; As ³ $A_{s,main}/2$) = 2 Æ 20

$prov\_A_{s,main}$ =	2*TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	12,56 cm²
n =	TAB("reinf/As"; n; Name=$A_{s,sel}$)	=	2

If $a_c \leq 0,5 * h_c$ closed horizontal or inclined links with $A_{s,link} \geq k_1 * A_{s,main}$ should be provided in addition to the main tension reinforcement.

$$\frac{a_c}{0,5 * h_c} = \underline{\mathbf{0,67 \leq 1}}$$

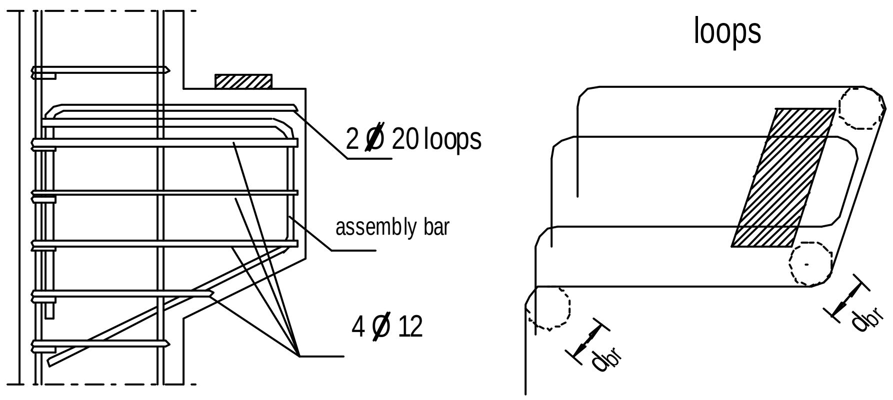
Horizontal links (Σ $A_{s,lnk} \geq A_{s,main}$)

$\Sigma A_{s,lnk}$=	$A_{s,main}$	=	8,25 cm²

Provide four H12 links

EN 1992-1-1 J.3.(2)

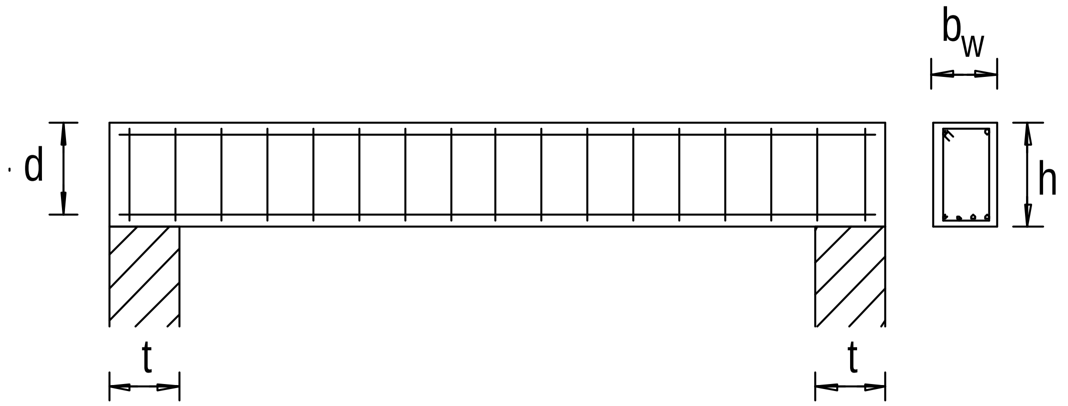
k_1 =			0,25
$req\_A_{s,lnk}$ =	$k_1 * A_{s,main}$	=	2,06 cm²
$d_{slnk,sel}$ =	SEL("reinf/As"; ds; $A_s \geq 0,5 * req\_A_{s,lnk}$; n=1)	=	12 mm
$A_{slnk,sel}$ =	SEL("reinf/As"; Name; $d_s = d_{slnk,sel}$; $As \geq \Sigma A_{s,lnk}/2$)	=	4 Ø 12
$prov\_A_{slnk}$ =	2* TAB("reinf/As"; As; Name=$A_{slnk,sel}$)	=	9,04 cm²
n =	TAB("reinf/As"; n; Name=$A_{slnk,sel}$)	=	4



horizontal link

Shear design of a rectangular section:

to EN 1992-1-1:2004



Section properties:

thickness t =	0,30 m
Beam width b_w =	0,30 m
height h =	0,42 m
effective depth d =	0,375 m

Actions:

Uniform load q_q =	9,41 kN/m
Uniform load q_g =	20,20 kN/m
Support reaction V_G =	65,25 kN
Support reaction V_Q =	30,39 kN

Materials, stresses and partial safety factors:

Concrete =	SEL("concrete/EC"; Name; f_{ck} £ 50)	=	C20/25
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	20,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a_{cc} =			1,00
f_{cd} =	$\frac{f_{ck} * a_{cc}}{1,5}$	=	13,33 N/mm²
g_G=			1,35
g_Q=			1,50

Shear design:

max_V =	$V_G * g_G + V_Q * g_Q$	=	133,67 kN
a_i =	MIN(1/2*h ;1/2*t)	=	0,15 m

Effective design shear force:

V_{Ed} =	max_V - ($q_g * g_G + q_q * g_Q$) * (a_i + d)	=	111,94 kN
v_1 =	$0,6 * \left(1 - \frac{f_{ck}}{250}\right)$	=	0,55
z =	0,9 * d	=	0,34 m

Limits EC2 for Q : 1 = cot Q = 2,5 (21,8°£ Q £ 45°)

provide cot Q x = 1,50

Consideration of the compressive strength of the diagonal concrete strut and its angle Q

Q = $\text{atan}\left(\frac{1}{x}\right)$ = 33,69 °

Is a coefficient taking account of the state of the stress in the compression chord.
For non-prestressed structures.

a_{cw} = 1,00

The angle of the shearing reinforcement is a = 90°

$V_{Rd,max} = \dfrac{a_{cw} * b_w * z * f_{cd} * v_1}{\left(\dfrac{1}{\tan(Q)} + \tan(Q)\right)} * 10^3$ = 345,14 kN

max_V/ $V_{Rd,max}$ = **0,39 < 1**

f_{ywd} = f_{yd} = 434,78 N/mm²

$req\_a_{sw} = \dfrac{V_{Ed} * 10^{-3}}{z * f_{ywd} * \dfrac{1}{\tan(Q)}} * 10^4$ = 5,05 cm²/m

Constructive shearing reinforcement

maximal spacing for links

$s_{t,max}$ = MIN(0,75*d; 0,60) = 0,28 m

$r_{w,min} = 0{,}08 * \dfrac{Ö\overline{f_{ck}}}{f_{yk}}$ = $0{,}72*10^{-3}$

a = 90,00°

$a_{sw,min}$ = $r_{w,min} * 10^4 * b_w$ * SIN(90) = 2,16 cm²/m

req_a_{sw} = MAX(req_a_{sw} ; $a_{sw,min}$) = 5,05 cm²/m

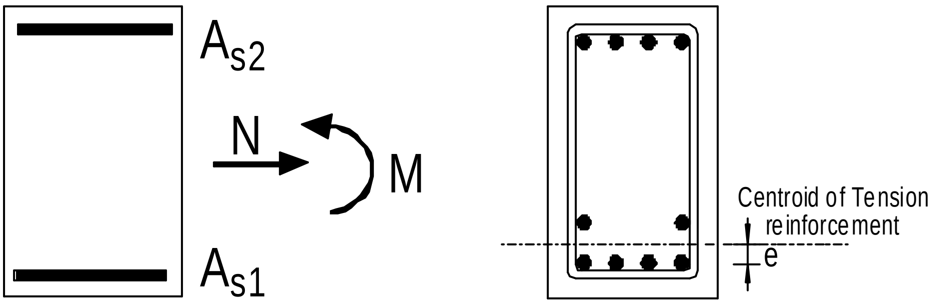
Provide links of size 8 mm (double-shear)

d_s =	SEL("reinf/AsArea"; ds;)	=	8,00 mm
a_s =	SEL("reinf/AsArea";Name;d_s=d_s;a_s ³ req_a_{sw}/2)	=	Æ 8 / e = 15
$prov\_a_{sw}$=	2 * TAB("reinf/AsArea"; as; Name=a_s)	=	6,70 cm²/m

req_a_{sw} / $prov\_a_{sw}$ = **0,75 < 1**

Flexural design of a rectangular section with compression reinforcement using the reinforcement ratio method

to EN 1992-1-1:2004



Section properties:

Width b = 0,25 m

Depth h = 0,75 m

Concrete cover c_{nom} = 0,035 m

Nominal axis distance a = 0,010 m

Bar size d_s = SEL("reinf/As"; ds;) = 28,0 mm

Bar size d_{s2} = SEL("reinf/As"; ds;) = 20,0 mm

Actions:

M_G= 250,00 kNm

M_Q= 240,00 kNm

N_G= -80,00 kN

N_Q= -60,00 kN

Materials and characteristic strengths:

Concrete = SEL("concrete/EC"; Name; $f_{ck} \le 50$) = C30/37

Steel = SEL("reinf/Steel"; Name;) = 500 S

f_{ck} = TAB("concrete/EC"; fck; Name=Concrete) = 30,00 N/mm²

f_{yk} = TAB("reinf/Steel"; βs; Name=Steel) = 500 MN/m²

$f_{tk,cal}$ = 525,00 N/mm²

Modulus of elasticity Steel E_s = 200000 N/mm²

Partial safety faktors:

γ_G = 1,35

γ_Q = 1,50

γ_s = 1,15

γ_c = 1,50

Design values and analysis:

$f_{yd} = f_{yk} / \gamma_S = 434{,}78$ N/mm²

$\varepsilon_{yd} = \frac{f_{yd}}{E_s} = 2{,}174 \cdot 10^{-3}$

$\varepsilon_{su} = 25{,}00 \cdot 10^{-3}$

$\alpha_{cc} = 1{,}00$

$f_{cd} = \frac{f_{ck} * \alpha_{cc}}{1{,}5} = 20{,}00$ N/mm²

$N_{Ed} = \gamma_G * N_G + \gamma_Q * N_Q = -198{,}00$ kN

$M_{Ed} = \gamma_G * M_G + \gamma_Q * M_Q = 697{,}50$ kNm

$d = h - c_{nom} - \frac{d_s}{2 * 10^3} - a = 0{,}691$ m

$z_{s1} = d - h / 2 = 0{,}316$ m

$M_{Ed,s} = ABS(M_{Ed}) - N_{Ed} * z_{s1} = 760{,}07$ kNm

$\mu_{Ed,s} = \frac{M_{Ed,s} * 10^{-3}}{b * d^2 * f_{cd}} = 0{,}318$

for C50/60 $\mu_{Ed,s,lim} = 0{,}296$

$\frac{\mu_{Ed,s}}{\mu_{Ed,s,lim}}$ = **<u>1,074 >1,0</u>**

$\Rightarrow$ compressive reinforcement is necessary!

ζ = TAB("reinf/Ecmy"; ζ; $\mu=\mu_{Ed,s,lim}$) = 0,813

ε_{c2} = TAB("reinf/Ecmy"; ε_{c2}; $\mu=\mu_{Ed,s}$)$*10^{-3}$ = $-3{,}50 \cdot 10^{-3}$

ε_{s1} = TAB("reinf/Ecmy"; ε_{s1}; $\mu=\mu_{Ed,s}$)$*10^{-3}$ = $3{,}58 \cdot 10^{-3}$

$lim\_M_{Ed,s} = \mu_{Ed,s,lim} * b * d^2 * f_{cd} * 10^3 = 706{,}67$ kNm

$\Delta M_{Ed,s} = M_{Ed,s} - lim\_M_{Ed,s} = 53{,}40$ kNm

$d_2 = c_{nom} + \frac{d_{s2}}{2 * 10^3} = 0{,}045$ m

$z = \zeta * d = 0{,}562$ m

$req\_A_{s1} = \frac{1}{f_{yd}} * \left(\frac{lim\_M_{Ed,s}}{z} + \frac{\Delta M_{Ed,s}}{d - d_2} + N_{Ed} \right) * 10 = 26{,}27$ cm²

Provide five 28 mm bars

$A_{s,prov}$ = SEL("reinf/As"; Name; $d_s = d_s$; As $\geq$ req_A_{s1}) = 5 Ø 28

$prov\_A_s$ = TAB("reinf/As"; As; Name=$A_{s,prov}$) = 30,79 cm²

$req\_A_{s1} / prov\_A_s$ = **<u>0,85 < 1</u>**

$req\_A_{s2} = \frac{1}{f_{yd}} * \frac{\Delta M_{Ed,s}}{d - d_2} * 10 = 1{,}90$ cm²

Provide two 12 mm bars

$A_{s,prov}$ =	SEL("reinf/As"; Name; $d_s = d_{s2}$; As ³ req_A_{s2})	=	2 Æ 20
prov_A_s =	TAB("reinf/As"; As; Name=$A_{s,prov}$)	=	6,28 cm²

req_A_{s2} / prov_A_s = **0,30 < 1**

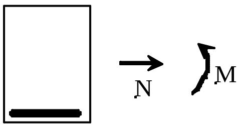
Check yield strength of tension reinforcement:

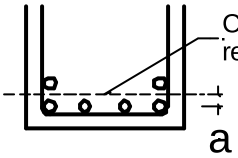
e_{yd} =			$2{,}17 \cdot 10^{-3}$
e_{s2} =	$(ABS(e_{c2}) + e_{s1}) * (d - d_2) / d - e_{s1}$	=	$3{,}04 \cdot 10^{-3}$

e_{s2} / e_{yd} = **1,40 > 1**

⇒ The maximum yield strength was achieved!

Flexural design of a rectangular section, K-method
to ENV 1992-1-1 !!!





Section properties:

Width b =	0,25 m
Depth h =	0,75 m
Concrete cover nom_c =	0,035 m
Nominal axis distance a =	0,020 m

Bar size d_s = SEL("reinf/As"; ds;) = 25,0 mm

Actions:

M_G=	250,00 kNm
M_Q=	150,00 kNm
N_G=	1,00 kN
N_Q=	0,00 kN

Materials and characteristic strengths:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²

Partial safety faktors:

g_S=	1,15
g_G=	1,35
g_Q=	1,50

Design values and analysis:

f_{yd} =	f_{yk} / g_S	=	434,78 kN/cm²
N_{sd} =	g_G * N_G + g_Q * N_Q	=	1,35 kN
M_{sd} =	g_G * M_G + g_Q * M_Q	=	562,50 kNm
d =	h - nom_c - d_s/10³ - a	=	0,670 m
z_{s1} =	d - h / 2	=	0,295 m
$M_{sd,s}$ =	ABS(M_{sd}) - N_{sd} * z_{s1}	=	562,10 kNm
k_d =	d * 100 / Ö($M_{sd,s}$ / b)	=	1,41

k_s =	TAB("reinf/kd"; ks1; Name=Concrete; kd=k_d)	=	2,830
z =	TAB("reinf/kd"; z; Name=Concrete; kd=k_d)	=	0,813
x =	TAB("reinf/kd"; x; Name=Concrete; kd=k_d)	=	0,450
e_{c2} =	TAB("reinf/kd"; e_{c2}; Name=Concrete; kd=k_d)	=	-3,500*10^{-3}
e_{s1} =	TAB("reinf/kd"; e_{s1}; Name=Concrete; kd=k_d)	=	4,270*10^{-3}

x=	ξ*d	=	0,301 m
z=	ζ*d	=	0,545 m

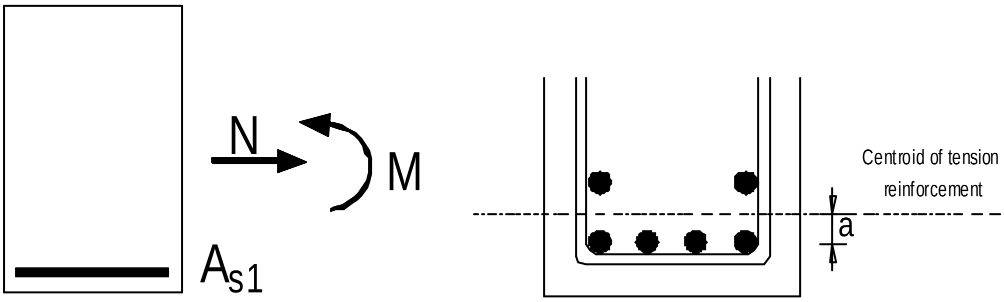
req_A_s=	$M_{sd,s}$/(d*100)*k_s+10*N_{sd}/f_{yd}	=	23,77 cm²

Provide five 25 mm bars

$A_{s,sel}$ =	SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s)	=	5 Æ 25
prov_A_s =	TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	24,54 cm²

req_A_s / prov_A_s = **0,97 £ 1**

Flexural design of a rectangular section
to EN 1992-1-1:2004



Section properties:

Width b =			0,30 m
Depth h =			0,75 m
Concrete cover c_{nom} =			0,035 m
Nominal axis distance a =			0,020 m
Bar size d_s =	SEL("reinf/As"; ds;)	=	25,0 mm

Actions:

M_G=	250,00 kNm
M_Q=	150,00 kNm
N_G=	-80,00 kN
N_Q=	-60,00 kN

Materials and characteristic strengths:

Concrete =	SEL("concrete/EC"; Name; $f_{ck} \leq$ 50)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; βs; Name=Steel)	=	500 MN/m²

Partial safety factors:

γ_G =	1,35
γ_Q =	1,50
γ_s =	1,15
γ_c =	1,50

Design values and analysis:

f_{yd} =	f_{yk} / γ_s	=	434,78 N/mm²
α_{cc} =			1,00
f_{cd} =	$\frac{f_{ck} * \alpha_{cc}}{1,5}$	=	20,00 N/mm²
N_{Ed} =	$\gamma_G * N_G + \gamma_Q * N_Q$	=	-198,00 kN
M_{Ed} =	$\gamma_G * M_G + \gamma_Q * M_Q$	=	562,50 kNm
d =	$h - c_{nom} - \frac{d_s}{2 * 10^3} - a$	=	0,683 m
z_{s1} =	d - h / 2	=	0,308 m

$M_{Ed,s}$ = ABS(M_{Ed}) - N_{Ed} * z_{s1} = 623,48 kNm

$\mu_{Ed,s} = \dfrac{M_{Ed,s} * 10^{-3}}{b * d^2 * f_{cd}}$ = 0,223

ζ = TAB("reinf/Ecmy"; ζ; μ=$\mu_{Ed,s}$) = 0,868

z = ζ * d = 0,593 m

$req\_A_{s1} = \dfrac{1}{f_{yd}} * \left(\dfrac{M_{Ed,s}}{z} + N_{Ed}\right) * 10$ = 19,63 cm²

Provide five 25 mm bars

$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_s; As ≥ req_A_{s1}) = 5 Ø 25

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 24,54 cm²

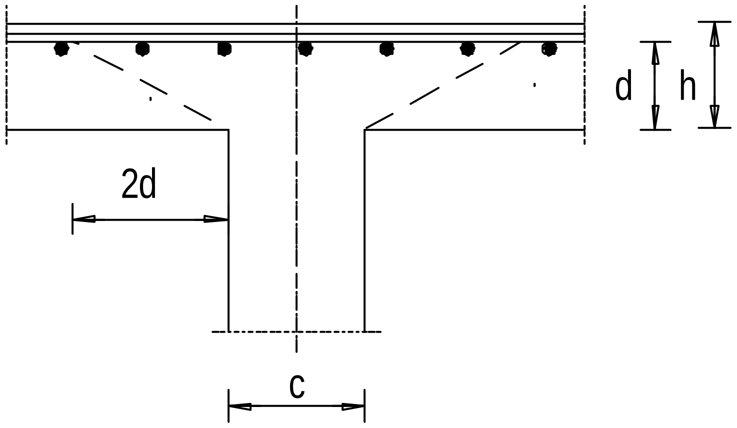
req_A_{s1} / prov_A_s = **0,80 < 1**

n = TAB("reinf/As"; n; Name=$A_{s,sel}$) = 5

Section width is appropriate!

CHECK = b-2*c_{nom} - (n*d_s + MAX(d_s ; 20)*(n-1))*10^{-3} = **0,00 ≥ 0**

Punching shear analysis in a flat slap without punching shear reinforcement :
to EN 1992-1-1:2004



Section properties:

Slap thickness h =			24,00 cm
Effective depth d_x =			19,00 cm
Effective depth d_y =			19,00 cm
column width b_x =			45,00 cm
column width b_y =			45,00 cm
d_{eff} =	$\frac{d_x + d_y}{2}$	=	19,00 cm

radial spacing of perimeters of shear reinforcement

s_r =	$0,75*d_{eff}$	=	14,25 mm
s_t =	$1,5*d_{eff}$	=	28,50 cm

Loads:

Shearing force for design:

V_{Ed} = 600 kN

Materials and stresses:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a_{cc} =			1,00
g_c=			1,50
f_{cd} =	$\frac{f_{ck} * a_{cc}}{g_c}$	=	20,00 N/mm²
g_G=			1,35
g_Q=			1,50

Top reinforcement in slab:

$reinf_x$ = SEL("reinf/AsArea"; Name;) = Æ 20 / e = 8.5

a_{sx} = TAB("reinf/AsArea"; a_s; Name=$reinf_x$) = 36,96 cm²/m

$reinf_y$ = SEL("reinf/AsArea"; Name;) = Æ 20 / e = 8.5

a_{sy} = TAB("reinf/AsArea"; a_s; Name=$reinf_y$) = 36,96 cm²/m

Design Calculation:

Punching shear analysis:

Critical perimeter:

$$u_1 = \frac{2 * (b_x + b_y + p * 2 * d_{eff})}{100} = 4{,}19 \text{ m}$$

$$\text{distance} = \frac{u_1 - (b_x + b_y) * 2 * 10^{-2}}{2 * p} = 0{,}38 \text{ m}$$

Shearing force per unit length along the critical perimeter:

for internal columns b = 1,15

$$n_{Ed} = \frac{b * V_{Ed} * 10^{-3}}{u_1 * d_{eff} * 10^{-2}} = 0{,}867 \text{ MN/m}^2$$

Maximum bearing capacity of slab per unit length without punching shear reinforcement:

$$k = \text{MIN}(1 + \ddot{O}\overline{\frac{200}{d_{eff} * 10}} \,;\, 2{,}0\,) = 2{,}0000$$

$$r_{lx} = \frac{a_{sx}}{b_x * d_x} = 0{,}0432$$

$$r_{ly} = \frac{a_{sy}}{b_y * d_y} = 0{,}0432$$

$$r_1 = \text{MIN}(\ddot{O}\,\overline{r_{lx} * r_{ly}}\,;0{,}02) = 0{,}0200$$

$$C_{Rd,c} = \frac{0{,}18}{g_c} = 0{,}12$$

$$n_{min} = 0{,}035 * \ddot{O}\,\overline{k^3} * \ddot{O}\,\overline{f_{ck}} = 0{,}54$$

k_1 = 0,10

<u>**Acceptance:**</u>

Concrete compressive stress at the centroidal axis due to axial loading and/or presstressing s_{cp}=0;

s_{cp} = 0,00 N/mm²

$$n_{Rd,c} = C_{Rd,c} * k * {}^3\ddot{O}\,\overline{(100 * r_1 * f_{ck})} + k_1 * s_{cp} = 0{,}940 \text{ MN/m}^2$$

$$n_{Rd,c,min} = n_{min} + k_1 * s_{cp} = 0{,}540 \text{ MN/m}^2$$

$$n_{Rd,c} = \text{MAX}(n_{Rd,c}\,;\,n_{Rd,c,min}) = 0{,}940 \text{ MN/m}^2$$

n_{Ed} / $n_{Rd,c}$ = **<u>0,92 < 1 !!!</u>**

$\Rightarrow$ **compressive reinforcement is not necessary, if $\nu_{Ed} / \nu_{Rd,c} \leq 1{,}0$**

ν is a stregth reduction factor for concrete in shear

ν = $0{,}6 * (1 - f_{ck}/250)$ = 0,528 MN/m²

for internal columns

u_0 = $2*(b_x + b_y) * 10$ = 1800 mm²

$\nu_{Ed,0}$ = $\dfrac{\beta * V_{Ed} * 10^3}{u_0 * d_{eff} * 10}$ = 2,0175 N/mm²

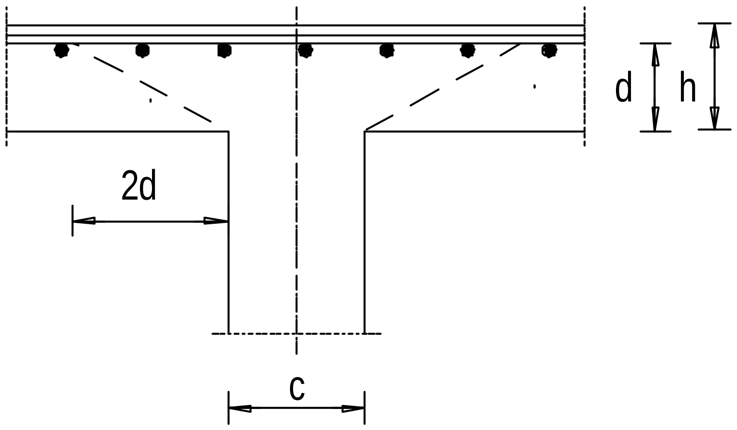
The maximum permissible shear force based on the face of the loaded area is given by the maximum shear resistance

$\nu_{Rd,max}$ = $0{,}5 * \nu * f_{cd}$ = 5,280 N/mm²

$\nu_{Ed,0} / \nu_{Rd,max}$ = **$0{,}38 \leq 1$**

Punching shear analysis in a flat slap:

to EN 1992-1-1:2004



Section properties:

Slap thickness h = 24,00 cm

Effective depth d_x = 19,00 cm

Effective depth d_y = 19,00 cm

column width b_x = 45,00 cm

column width b_y = 45,00 cm

$$d_{eff} = \frac{d_x + d_y}{2} = 19{,}00 \text{ cm}$$

radial spacing of perimeters of shear reinforcement

s_r = $0{,}75 \cdot d_{eff}$ = 14,25 mm

s_t = $1{,}5 \cdot d_{eff}$ = 28,50 cm

Loads:

Shearing force for design:

V_{Ed} = 809 kN

Materials and stresses:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a_{cc} =			1,00
g_c=			1,50

$$f_{cd} = \frac{f_{ck} * a_{cc}}{g_c} = 20{,}00 \text{ N/mm²}$$

g_G= 1,35

g_Q= 1,50

Top reinforcement in slab:

A_{sx} =	SEL("reinf/AsArea"; Name;)	=	Ø 20 / e = 8.5
a_{sx} =	TAB("reinf/AsArea"; a_s; Name=A_{sx})	=	36,96 cm²/m
A_{sy} =	SEL("reinf/AsArea"; Name;)	=	Ø 20 / e = 8.5
a_{sy} =	TAB("reinf/AsArea"; a_s; Name=A_{sy})	=	36,96 cm²/m

Design Calculation:

d_{eff} = $(d_x + d_y) / 2$ = 19,00 cm

Punching shear analysis:

Critical perimeter:

$$u_1 = \frac{2 * (b_x + b_y + \pi * 2 * d_{eff})}{100} = 4{,}19 \text{ m}$$

$$\text{distance} = \frac{u_1 - (b_x + b_y) * 2 * 10^{-2}}{2 * \pi} = 0{,}38 \text{ m}$$

Shearing force per unit length along the critical perimeter:

for internal columns β = 1,15

$$\nu_{Ed} = \frac{\beta * V_{Ed} * 10^{-3}}{u_1 * d_{eff} * 10^{-2}} = 1{,}169 \text{ MN/m²}$$

Maximum bearing capacity of slab per unit length without punching shear reinforcement:

$$k = MIN(1 + \sqrt{\frac{200}{d_{eff} * 10}} ; 2{,}0) = 2{,}0000$$

$$\rho_{lx} = \frac{a_{sx}}{b_x * d_x} = 0{,}0432$$

$$\rho_{ly} = \frac{a_{sy}}{b_y * d_y} = 0{,}0432$$

$$\rho_1 = MIN(\sqrt{\rho_{lx} * \rho_{ly}} ; 0{,}02) = 0{,}0200$$

$$C_{Rd,c} = \frac{0{,}18}{\gamma_c} = 0{,}12$$

$$\nu_{min} = 0{,}035 * \sqrt{k^3} * \sqrt{f_{ck}} = 0{,}54$$

k_1 = 0,10

Acceptance:

Concrete compressive stress at the centroidal axis due to axial loading and/or presstressing σ_{cp}=0;

σ_{cp} = 0,00 N/mm²

$$\nu_{Rd,c} = C_{Rd,c} * k * \sqrt[3]{(100 * \rho_1 * f_{ck})} + k_1 * \sigma_{cp} = 0{,}940 \text{ MN/m²}$$

$$\nu_{Rd,c,min} = \nu_{min} + k_1 * \sigma_{cp} = 0{,}540 \text{ MN/m²}$$

$$\nu_{Rd,c} = MAX(\nu_{Rd,c} ; \nu_{Rd,c,min}) = 0{,}940 \text{ MN/m²}$$

$\nu_{Ed} / \nu_{Rd,c}$ = **<u>1,24 > 1 !!!</u>**

⇒ compressive reinforcement is necessary!

ν = $0{,}6 * (1- f_{ck} /250)$ = 0,528 MN/m²

for internal columns

u_0= $2*(b_x + b_y) * 10$ = 1800 mm²

$\nu_{\mathbf{Ed,0}} = \dfrac{\beta * V_{Ed} * 10^3}{u_0 * d_{eff} * 10}$ = 2,7203 N/mm²

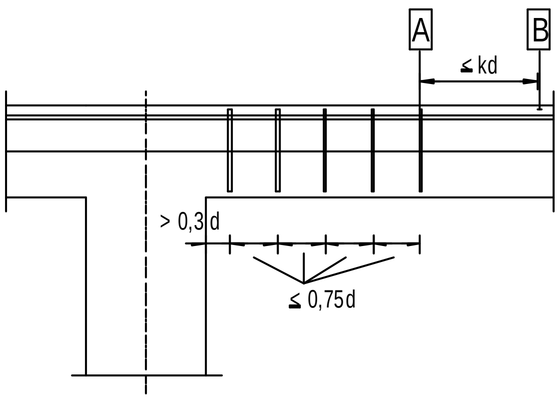
The maximum permissible shear force based on the face of the loaded area is given by the maximum shear resistance

$\nu_{\mathbf{Rd,max}}$ = $0{,}5 * \nu * f_{cd}$ = 5,280 N/mm²

$\nu_{Ed,0} / \nu_{\mathbf{Rd,max}}$ = **<u>0,52 ≤ 1</u>**

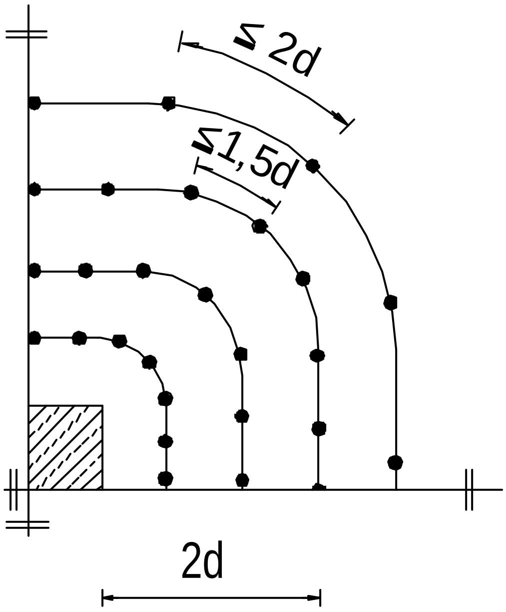
Reinforcement for punching shear:

$f_{ywd,ef}$ = $250 + 0{,}25 * d_{eff} * 10$ = 297,50 N/mm²



$\boxed{A}$ perimeter, at which reinforcement is required.

$\boxed{B}$ outer perimeter uout,ef at which reinforcement is not required.



s_r radial distance link of punching shear reinforcement

$s_r =$ $0{,}75 \cdot d_{eff} \cdot 10$ = 142,50 mm

s_t tangential distance link of punching shear reinforcement

$s_t =$ $1{,}5 \cdot d_{eff} \cdot 10$ = 285,00 mm

first row:

distance $a_1 =$ $0{,}5 \cdot d_{eff}$ = 9,50 cm

$$u_{cont,1} = \left(\left(b_x + b_y\right) \cdot 2 + p \cdot a_1 \cdot 2\right) \cdot 10^{-2} = 2{,}40 \text{ m}$$

$$n_{Ed,1} = \frac{b \cdot V_{Ed} \cdot 10^{-3}}{u_{cont,1} \cdot d_{eff} \cdot 10^{-2}} = 2{,}04 \text{ N/mm}^2$$

$$A_{sw,1} = \frac{n_{Ed,1} - 0{,}75 \cdot n_{Rd,c}}{1{,}5 \cdot \left(\frac{d_{eff} \cdot 10}{s_r}\right) \cdot f_{ywd,ef} \cdot \left(\frac{1}{u_1 \cdot 10^3 \cdot d_{eff} \cdot 10}\right) \cdot \sin(90)} \cdot 10^{-2} = 17{,}86 \text{ cm}^2$$

A check must also be made the calculated reinforcement satisfies the minimum requirement that:

The angle of the Punching shear reinforcement is a = 90°

$$A_{sw,min} = \frac{0{,}08 \cdot \frac{\sqrt{f_{ck}}}{f_{yk}}}{\left(1{,}5 \cdot \sin(90) + \cos(90)\right) / \left(s_r \cdot 10^{-3} \cdot u_{cont,1}\right)} \cdot 10^4 = 2{,}00 \text{ cm}^2$$

$A_{sw,1} =$ MAX($A_{sw,1}$; $A_{sw,min}$) = 17,86 cm²

$$\text{Number of link-legs } n_1 = \frac{u_{cont,1}}{1{,}5 \cdot d_{eff} \cdot 10^{-2}} + 0{,}5 = 9$$

$d_{s,sel}$ =	SEL("reinf/As"; ds;)	=	14 mm
A_s =	SEL("reinf/As"; Name; ds=$d_{s,sel}$;n³ n_1;A_s³ $A_{sw,1}$)	=	12 Æ 14
prov_$A_{sw,1}$ =	TAB("reinf/As"; As; Name=A_s)	=	18,47 cm²

secand row:

distance a_2 = $0{,}75 \cdot d_{eff}$ = 14,25 cm

$$u_{cont,2} = ((b_x + b_y) \cdot 2 + p \cdot (a_1 + a_2) \cdot 2) \cdot 10^{-2} = 3{,}29 \text{ m}$$

$$n_{Ed,2} = \frac{b \cdot V_{Ed} \cdot 10^{-3}}{u_{cont,2} \cdot d_{eff} \cdot 10^{-2}} = 1{,}49 \text{ N/mm}^2$$

$$A_{sw,2} = \frac{n_{Ed,2} - 0{,}75 \cdot n_{Rd,c}}{1{,}5 \cdot \left(\frac{d_{eff} \cdot 10}{s_r}\right) \cdot f_{ywd,ef} \cdot \left(\frac{1}{u_1 \cdot 10^3 \cdot d_{eff} \cdot 10}\right) \cdot \sin(90)} \cdot 10^{-2} = 10{,}50 \text{ cm}^2$$

A check must also be made the calculated reinforcement satisfies the minimum requirement that:
The angle of the Punching shear reinforcement is a = 90°

$$A_{sw,min} = \frac{0{,}08 \cdot \frac{\sqrt{f_{ck}}}{f_{yk}}}{(1{,}5 \cdot \sin(90) + \cos(90)) / \left(s_r \cdot 10^{-3} \cdot u_{cont,2}\right)} \cdot 10^4 = 2{,}74 \text{ cm}^2$$

$A_{sw,2}$ = MAX($A_{sw,2}$; $A_{sw,min}$) = 10,50 cm²

$$\text{Number of link-legs } n_2 = \frac{u_{cont,2}}{1{,}5 \cdot d_{eff} \cdot 10^{-2}} + 0{,}5 = 12$$

$d_{s,sel}$ =	SEL("reinf/As"; ds;)	=	14 mm
A_s =	SEL("reinf/As"; Name; ds=$d_{s,sel}$; A_s³ $A_{sw,2}$;n³ n_2)	=	12 Æ 14
prov_$A_{sw,2}$ =	TAB("reinf/As"; As; Name=A_s)	=	18,47 cm²

check

$A_{sw,1}$ / prov_$A_{sw,1}$ = **<u>0,97 < 1</u>**

$A_{sw,2}$ / prov_$A_{sw,2}$ = **<u>0,57 < 1</u>**

outer perimeter $u_{out,ef}$ at which reinforcement is not required.

$$u_{out,ef} = \frac{b \cdot V_{Ed} \cdot 10^{-3}}{n_{Rd,c} \cdot d_{eff} \cdot 10^{-2}} = 5{,}21 \text{ m}$$

$$\text{Distance}_{out,ef} = \frac{u_{out,ef} - (b_x + b_y) \cdot 2 \cdot 10^{-2}}{2 \cdot p} = 0{,}54 \text{ m}$$

Factor k = 1,50

kd = $k*d_{eff}$ = 28,50 cm

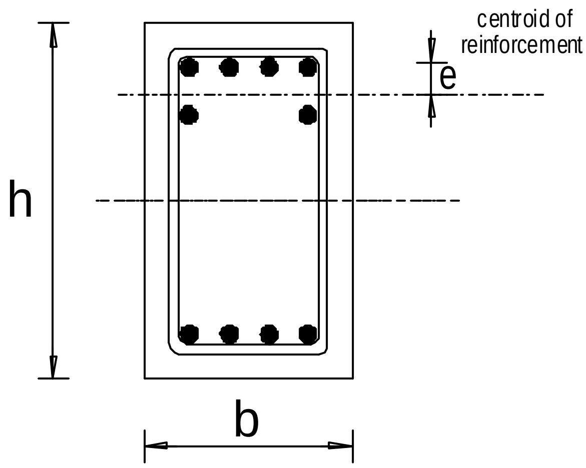
remained distance = $Distance_{out,ef}$ -kd/100-(a_1 + a_2)/100 = 0,02 cm

$$n_{Ed,a} = \frac{b * V_{Ed} * 10^{-3}}{u_{out,ef} * d_{eff} * 10^{-2}} = 0,94 \text{ N/mm}^2$$

$n_{Ed,a} / n_{Rd,c}$ = **1,00 < 1**

Rectangular section with shear and torsion:

to EN 1992-1-1:2004



System:

thickness t =			0,30 m
Beam width b=			0,40 m
Beam height h=			0,60 m
nom_c=			0,042 m
distance e =			0,032 m
From live load q_q=			9,41 kN/m
From dead load q_g=			20,20 kN/m
Support reaction V_G=			150,00 kN
Support reaction V_Q =			89,00 kN
Torsional moment T_{Ed}=			45,00 kNm
Area A =	h*b	=	0,24 m²
perimeter u =	2*(h+b)	=	2,00 m
effective wall thickness $t_{ef,1}$ =	A/u	=	0,12 m
from flexural design prov_$A_{S,M}$ =			12,50 cm²

Materials, stresses and partial safety factors:

Concrete =	SEL("concrete/EC"; Name;f_{ck} ≤ 50)	=	C25/30
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	25,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; βs; Name=Steel)	=	500 MN/m²
f_{ctm} =	TAB("concrete/EC"; fctm; Name=Concrete)	=	2,60 N/mm ²
f_{ctk} =	TAB("concrete/EC"; fctk05; Name=Concrete)	=	1,80 N/mm²
d =	h-nom_c-e	=	0,526 m
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
α_{ct} =			1,00
α_{cc} =			1,00
f_{ctd} =	$\frac{f_{ctk} * \alpha_{cc}}{1,5}$	=	1,20 N/mm²
f_{cd} =	$\frac{f_{ck} * \alpha_{cc}}{1,5}$	=	16,67 N/mm²
γ_G=			1,35
γ_Q=			1,50
γ_c=			1,50

Shear design:

d = h-nom_c - e = 0,526 m

max_V = $V_G * g_G + V_Q * g_Q$ = 336 kN

a_i = MIN(1/2*h ;1/2*t) = 0,15 m

maßgebende Querkraft:

V_{Ed} = max_V - $(q_g * g_G + q_q * g_Q) * (a_i + d)$ = 308,02 kN

z = 0,9 * d = 0,47 m

Limits EC2 for Q : 1 = cot Q = 2,5 (21,8°£ Q £ 45°)

provide cot Q x = 1,00

Consideration of the compressive strength of the diagonal concrete strut and its angle Q

$$Q = \text{atan}\left(\frac{1}{x}\right) = 45{,}00\ °$$

smallest beam width b_w = b = 0,40 m

b_k = $b - t_{ef,1}$ = 0,28 m

h_k = $h - t_{ef,1}$ = 0,48 m

A_k Area enclosed by the centre lines of the connecting walls including the inner hollow areas

A_k = $h_k * b_k$ = 0,13 m²

u_k = $2*(b_k + h_k)$ = 1,52 m

$$\text{Normal Concrete } v_1 = 0{,}6 * \left(1 - \frac{f_{ck}}{250}\right) = 0{,}54$$

$$k = \text{MIN}\left(1 + Ö\frac{200}{d * 10^3}\ ;\ 2\right) = 1{,}62$$

$$r_1 = \text{MIN}\left(\frac{prov\_A_{S,M}}{b_w * d * 10^4}\ ;\ 0{,}02\right) = 0{,}006$$

recommened Value

k_1 = 0,15

$C_{Rd,c}$ = $0{,}18/g_c$ = 0,12

<u>**Acceptance:**</u>

Concrete compressive stress at the centroidal axis due to axial loading and/or presstressing s_{cp}=0;

s_{cp} = 0,00 N/mm²

$$V_{Rd,c} = \left(C_{Rd,c} * k * \sqrt[3]{100 * r_1 * f_{ck}} + k_1 * s_{cp}\right) * b_w * d * 10^3 = 100{,}87\ \text{kN}$$

for retangle (l ³ t)

l = MAX(b_k ;h_k) = 0,480 m

t = MIN(b_k ;h_k) = 0,280 m

l/t = 1,714 m

h_R = TAB("concrete/RFactor"; h ; e=l/t) = 0,210

a_R = TAB("concrete/RFactor"; a ; e=l/t) = 4,219

I_T = $h_R * l * t^3$ = 2,213*10⁻³ m⁴

$$W_T = \frac{l * t^2}{a_R} = 8{,}920*10^{-3}\ m^3$$

$T_{Rd,c}$ = $f_{ctd} * W_T * 10^3$ = 10,70 kN/m²

It is only the minimum of reininforcement necessary, if the following proof are sucessfully

$$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}} = \underline{\mathbf{7{,}26 < 1}}$$

Is a coefficient taking account of the state of the stress in the compression chord.
For non-prestressed structures.

a_{cw} = 1,00

The angle of the shearing reinforcement is a = 90°

$$V_{Rd,max} = \frac{a_{cw} * b_w * z * f_{cd} * v_1}{\left(\frac{1}{\tan(Q)} + \tan(Q)\right)} * 10^3 = 846{,}17 \text{ kN}$$

$$max\_V / V_{Rd,max} = \underline{\mathbf{0{,}40 < 1}}$$

f_{ywd} = f_{yd} = 434,78 N/mm²

$$req\_a_{sw,V} = \frac{V_{Ed}}{z * f_{ywd} * \sin(90) * \left(\frac{1}{\tan(Q)} + \frac{1}{\tan(90)}\right)} * 10 = 15{,}07 \text{ cm}^2/\text{m}$$

Minimum shear reinforcement ratio

$$r_{w,min} = 0{,}08 * \frac{\sqrt{f_{ck}}}{f_{yk}} = 0{,}80*10^{-3}$$

maximal spacing for links

$s_{t,max}$ = MIN(0,75*d; 0,60) = 0,39 m

$a_{sw,min}$ = $r_{w,min} * 10^4 * b_w$ * SIN(90) = 3,20 cm²/m

$req\_a_{sw,V}$ = MAX($req\_a_{sw,V}$; $a_{sw,min}$) = 15,07 cm²/m

Design for torsion:

v = v_1 = 0,54

$T_{Rd,max}$ = $v * a_{cw} * f_{cd} * 2 * A_k * t_{ef,1}$ * SIN(Q) * COS(Q)*10^3 = 140,43 kN

$T_{Ed} / T_{Rd,max}$ = $\underline{\mathbf{0{,}32 < 1}}$

tension reinforcement as a result of torsion

$$req\_a_{sl,T} = \frac{T_{Ed} * 10}{2 * A_k * f_{yd} * \tan(Q)} = 3{,}98 \text{ cm}^2/\text{m}$$

links as a result of torsion

$$req\_a_{sw,T} = \frac{T_{Ed} * 10 * \tan(Q)}{2 * A_k * f_{yd}} = 3{,}98 \text{ cm}^2/\text{m}$$

$T_{Ed} / T_{Rd,max} + V_{Ed} / V_{Rd,max}$ = $\underline{\mathbf{0{,}68 < 1}}$

reinforcement links:

req_a_{sw} = $2 * req\_a_{sw,T} + req\_a_{sw,V}$ = **23,03 cm²/m**

Provide links of size 12 mm (double-shear)

d_s = SEL("reinf/AsArea"; ds;) = 12,00 mm

a_s = SEL("reinf/AsArea";Name;d_s=d_s;a_s ³ req_a_{sw}/2) = Æ 12 / e = 9.5

$prov\_a_{sw}$= 2*TAB("reinf/AsArea"; as; Name=a_s) = 23,80 cm²/m

$req\_a_{sw} / prov\_a_{sw}$ = **0,97 < 1**

torsion reinforcement longitudinal bars:

req_A_{Sl} = $prov\_A_{S,M} + req\_a_{sl,T} * u_k /3$ = 14,52 cm²

$A_{s,sel}$ = SEL("reinf/As"; Name; As ³ req_A_{Sl}) = 5 Æ 20

$prov\_A_s$ = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 15,71 cm²

provide five bars 20 mm

at the side:

$req\_A_{Sl,T}$ = $req\_a_{sl,T} * u_k /3$ = 2,02 cm²

$A_{s,sel}$ = SEL("reinf/As"; Name; As ³ $req\_A_{Sl,T}$) = 2 Æ 14

$prov\_A_s$ = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 3,08 cm²

provide two bars 14 mm

side of compression

$req\_A_{Sl,T}$ = $req\_a_{sl,T} * u_k /3$ = 2,02 cm²

$A_{s,sel}$ = SEL("reinf/As"; Name; As ³ $req\_A_{Sl,T}$) = 2 Æ 14

$prov\_A_s$ = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 3,08 cm²

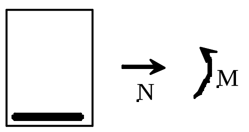
provide two bars 14 mm

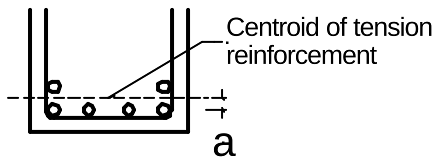
minimum reinforcement longitudinal bars :

$$A_{Sl,min} = MAX(0,26 * \frac{f_{ctm}}{f_{yk}} * b * d * 10^4 ; 0,0013 * b * d * 10^4) = 2,84\ cm^2$$

$$\frac{A_{Sl,min}}{req\_a_{sl,T} * u_k} = \mathbf{0,47\ £1}$$

Rectangular section using the kh-method
to ENV 1992-1-1 !!!





Section properties:

Width b =	0,25 m
Depth h =	0,75 m
Concrete cover nom_c =	0,035 m
Distance between centre of tension and lowest bar a =	0,020 m

Bar size d_s =	SEL("reinf/As"; ds;)	=	25,0 mm

Loads:

M_G=	250,00 kNm
M_Q=	150,00 kNm
N_G=	1,00 kN
N_Q=	0,00 kN

Materials and stresses:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²

Partial safety faktors:

g_s=	1,15
g_G=	1,35
g_Q=	1,50

Design Calculation:

f_{yd} =	f_{yk} / g_s	=	434,78 kN/cm²
N_{sd} =	g_G * N_G + g_Q * N_Q	=	1,35 kN
M_{sd} =	g_G * M_G + g_Q * M_Q	=	562,50 kNm
d =	h - nom_c - d_s/10³ - a	=	0,670 m
z_{s1} =	d - h / 2	=	0,295 m
$M_{sd,s}$ =	ABS(M_{sd}) - N_{sd} * z_{s1}	=	562,10 kNm
k_d =	d * 100 / Ö($M_{sd,s}$ / b)	=	1,41
k_s =	TAB("reinf/kd"; ks1; Name=Concrete; kd=k_d)	=	2,830
z =	TAB("reinf/kd"; z; Name=Concrete; kd=k_d)	=	0,813
x =	TAB("reinf/kd"; x; Name=Concrete; kd=k_d)	=	0,450
e_{c2} =	TAB("reinf/kd"; e_{c2}; Name=Concrete; kd=k_d)	=	$-3,500*10^{-3}$
e_{s1} =	TAB("reinf/kd"; e_{s1}; Name=Concrete; kd=k_d)	=	$4,270*10^{-3}$
x=	ξ*d	=	0,301 m
z=	ζ*d	=	0,545 m

req_A_s= $M_{sd,s}$/(d*100)*k_s+10*N_{sd}/f_{yd} = 23,77 cm²

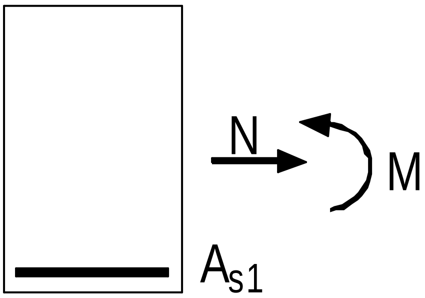
Provide five 25 mm bars

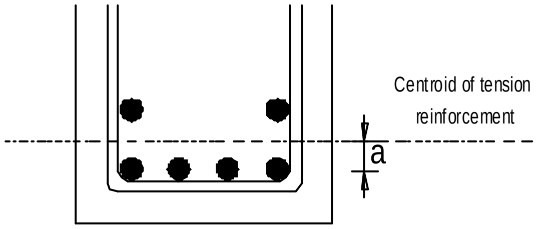
$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s) = 5 Æ 25

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 24,54 cm²

req_A_s / prov_A_s = **0,97 £ 1**

Rectangular section with the Omega-method:
to EN 1992-1-1:2004





Section properties:

Width b =		0,30 m
Depth h =		0,75 m
Concrete cover c_{nom} =		0,035 m
Distance between centre of tension and lowest bar a =		0,020 m
Bar size d_s = SEL("reinf/As"; ds;)	=	25,0 mm

Loads:

M_G=	250,00 kNm
M_Q=	150,00 kNm
N_G=	-80,00 kN
N_Q=	-60,00 kN

Materials and stresses:

Concrete =	SEL("concrete/EC"; Name;f_{ck} £ 50)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
a_{cc} =			1,00

Partial safety faktors:

g_G =	1,35
g_Q =	1,50
g_s =	1,15
g_c =	1,50

Design Calculation:

f_{yd} = f_{yk} / g_s = 434,78 kN/cm²

f_{cd} = $\frac{f_{ck} * a_{cc}}{g_c}$ = 20,00 N/mm²

N_{Ed} = $g_G * N_G + g_Q * N_Q$ = -198,00 kN

M_{Ed} = $g_G * M_G + g_Q * M_Q$ = 562,50 kNm

d = $h - c_{nom} - \frac{d_s * 10^{-3}}{2} - a$ = 0,683 m

z_{s1} = $d - h / 2$ = 0,308 m

$M_{Ed,s}$ = $ABS(M_{Ed}) - N_{Ed} * z_{s1}$ = 623,48 kNm

$m_{Ed,s} = \dfrac{M_{Ed,s} * 10^{-3}}{b * d^{2} * f_{cd}}$ = 0,223

w = TAB("reinf/Ecmy"; w; $m = m_{Ed,s}$) = 0,257

$req\_A_s = \dfrac{w * d * b * f_{cd} + N_{Ed} * 10^{-3}}{f_{yd}} * 10^{4}$ = 19,67 cm²

$A_{s,sel}$ = SEL("reinf/As"; Name; $d_s = d_s$; As ^{3} req_A_s) = 5 Æ 25

$prov\_A_s$ = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 24,54 cm²

req_A_s / $prov\_A_s$ = **0,80 < 1**

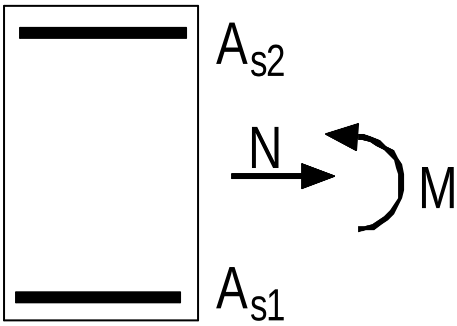
n = TAB("reinf/As"; n; Name=$A_{s,sel}$) = 5

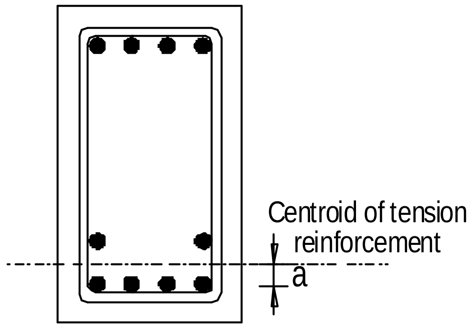
the width of the retangular is enough (add the legs of links, if exist)

CHECK = $b-2*c_{nom} - (n*d_s + MAX(d_s ; 20)*(n-1))*10^{-3}$ = **0,00 ^{3} 0**

Rectangular section with compression reinforcement (omega-method)

to EN 1992-1-1:2004





Section properties:

Width b =		0,25 m
Depth h =		0,75 m
Concrete cover c_{nom} =		0,035 m
Distance between centre of tension and lowest bar a =		0,010 m
Bar size d_s =	SEL("reinf/As"; ds;) =	28,0 mm
Bar size d_{s2} =	SEL("reinf/As"; ds;) =	20,0 mm

Loads:

M_G=	250,00 kNm
M_Q=	240,00 kNm
N_G=	-80,00 kN
N_Q=	-60,00 kN

Materials and stresses:

Concrete =	SEL("concrete/EC"; Name;f_{ck} £ 50) =	C30/37
Steel =	SEL("reinf/Steel"; Name;) =	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete) =	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel) =	500 MN/m²
Modulus of elasticity Steel E_s =		200000 N/mm²

Partial safety faktors:

g_G =	1,35
g_Q =	1,50
g_s =	1,15
g_c =	1,50

Design Calculation:

$f_{yd} = f_{yk} / g_s = 434{,}78$ N/mm²

$$e_{yd} = \frac{f_{yd}}{E_s} = 2{,}174 \cdot 10^{-3}$$

a_{cc} = 1,00

$$f_{cd} = \frac{f_{ck} * a_{cc}}{1{,}5} = 20{,}00 \text{ N/mm²}$$

$N_{Ed} = g_G * N_G + g_Q * N_Q = -198{,}00$ kN

M_{Ed} = $g_G * M_G + g_Q * M_Q$ = 697,50 kNm

d = $h - c_{nom} - \frac{d_s}{2 * 10^3} - a$ = 0,691 m

z_{s1} = $d - h / 2$ = 0,316 m

$M_{Ed,s}$ = $ABS(M_{Ed}) - N_{Ed} * z_{s1}$ = 760,07 kNm

for C50/60 $m_{Ed,s,lim}$ = 0,296

$m_{Ed,s}$ = $\frac{M_{Ed,s} * 10^{-3}}{b * d^2 * f_{cd}}$ = 0,318

$\frac{m_{Ed,s}}{m_{Ed,s,lim}}$ = **$1{,}07$ £ 1**

Þ compressive reinforcement is necessary!

d_2 = $c_{nom} + \frac{d_{s2}}{2 * 10^3}$ = 0,045 m

Difference m_D = $m_{Ed,s} - m_{Ed,s,lim}$ = 0,022

z = TAB("reinf/Ecmy"; z; $m=m_{Ed,s,lim}$) = 0,813

e_{c2} = TAB("reinf/Ecmy"; e_{c2}; $m=m_{Ed,s}$)$*10^{-3}$ = $-3{,}50*10^{-3}$

e_{s1} = TAB("reinf/Ecmy"; e_{s1}; $m=m_{Ed,s}$)$*10^{-3}$ = $3{,}58*10^{-3}$

$w_{1,lim}$ = $\frac{m_{Ed,s,lim}}{z}$ = 0,364

w_D = $\frac{m_D}{1 - \frac{d_2}{d}}$ = 0,024

req_A$_{s1}$ = $\frac{(w_{1,lim} + w_D) * d * b * f_{cd} + N_{Ed} * 10^{-3}}{f_{yd}} * 10^4$ = 26,28 cm²

Provide five 28 mm bars

$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_s; As ³ req_A$_{s1}$) = 5 Æ 28

prov_A$_s$ = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 30,79 cm²

req_A$_{s1}$ / prov_A$_s$ = **0,85 < 1**

Provide two 12 mm bars

w_2 = w_D = 0,024

$req\_A_{s2} = \dfrac{w_2 * d * b * f_{cd}}{f_{yd}} * 10^4$ = 1,91 cm²

$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_{s2}; As ³ req_A_{s2}) = 2 Æ 20

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 6,28 cm²

req_A_{s2} / prov_A_s = **0,30 < 1**

Check tensile strain yield:

e_{yd} = 2,17*10^{-3}

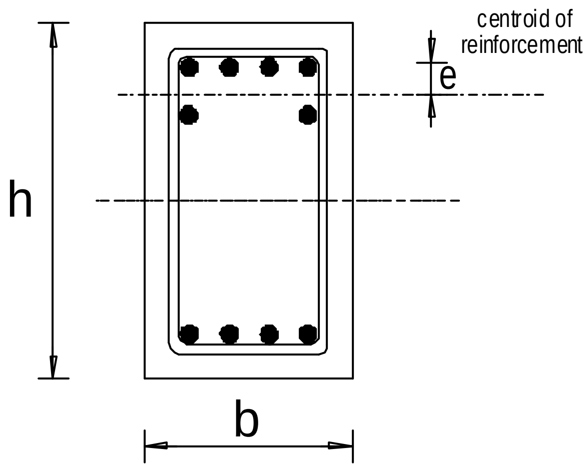
e_{s2} = (ABS(e_{c2}) + e_{s1}) * (d - d_2) / d - e_{s1} = 3,04*10^{-3}

e_{s2} / e_{yd} = **1,40 > 1**

⇒ Strain yield limit achieved!

Rectangular section with shear and torsion:

to EN 1992-1-1:2004



System:

thickness t =			0,30 m
Beam width b=			0,40 m
Beam height h=			0,60 m
nom_c=			0,042 m
distance e =			0,032 m
From live load q_q=			9,41 kN/m
From dead load q_g=			20,20 kN/m
Support reaction V_G=			150,00 kN
Support reaction V_Q =			89,00 kN
Torsional moment T_{Ed}=			45,00 kNm
Area A =	h*b	=	0,24 m²
perimeter u =	2*(h+b)	=	2,00 m
effective wall thickness $t_{ef,1}$ =	A/u	=	0,12 m
from flexural design prov_$A_{S,M}$ =			12,50 cm²

Materials, stresses and partial safety factors:

Concrete =	SEL("concrete/EC"; Name;f_{ck} £ 50)	=	C25/30
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	25,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{ctm} =	TAB("concrete/EC"; fctm; Name=Concrete)	=	2,60 N/mm ²
f_{ctk} =	TAB("concrete/EC"; fctk05; Name=Concrete)	=	1,80 N/mm²
d =	h-nom_c-e	=	0,526 m
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a **ct** =			1,00
a **cc** =			1,00

Is a coefficient taking account of the state of the stress in the compression chord.
For non-prestressed structures.

a $_{cw}$ = 1,00

$$f_{ctd} = \frac{f_{ctk} * a_{cc}}{1,5} = 1,20 \text{ N/mm}^2$$

$f_{cd} = \frac{f_{ck} * a_{cc}}{1,5}$ = 16,67 N/mm²

g_G= 1,35

g_Q= 1,50

g_c= 1,50

Shear design:

d =	h-nom_c - e	=	0,526 m
max_V =	$V_G * g_G + V_Q * g_Q$	=	336 kN
a_i =	MIN(1/2*h ;1/2*t)	=	0,15 m

maßgebende Querkraft:

V_{Ed} =	max_V - $(q_g * g_G + q_q * g_Q)$ * $(a_i + d)$	=	308,02 kN
z =	0,9 * d	=	0,47 m

Limits EC2 for Q : 1 = cot Q = 2,5 (21,8°£ Q £ 45°)

provide cot Q x = 1,00

Consideration of the compressive strength of the diagonal concrete strut and its angle Q

Q =	$\operatorname{atan}\left(\frac{1}{x}\right)$	=	45,00 °
smallest beam width b_w =	b	=	0,40 m
b_k =	$b\text{-}t_{ef,1}$	=	0,28 m
h_k =	$h\text{-}t_{ef,1}$	=	0,48 m

A_k Area enclosed by the centre lines of the connecting walls including the inner hollow areas

A_k =	$h_k * b_k$	=	0,13 m²
u_k =	$2*(b_k + h_k)$	=	1,52 m
Normal Concrete v_1 =	$0,6 * \left(1 - \frac{f_{ck}}{250}\right)$	=	0,54
k =	MIN$\left(1 + Ö\frac{200}{d * 10^3}\right.$; 2)	=	1,62
r_1 =	MIN$\left(\frac{prov_A_{S,M}}{b_w * d * 10^4}\right.$; 0,02)	=	0,006

recommened Value

k_1 =			0,15
$C_{Rd,c}$ =	$0,18/g_c$	=	0,12

Acceptance:

Concrete compressive stress at the centroidal axis due to axial loading and/or presstressing s_{cp}=0;

s_{cp} = 0,00 N/mm²

$V_{Rd,c} = \left(C_{Rd,c} * k * {}^3Ö\overline{100 * r_1 * f_{ck}} + k_1 * s_{cp}\right) * b_w * d * 10^3$ = 100,87 kN

for retangle (l ³ t)

$l =$ MAX(b_k;h_k) = 0,480 m

$t =$ MIN(b_k;h_k) = 0,280 m

l/t = 1,714

$h_R =$ TAB("concrete/RFactor"; h; e=l/t) = 0,210

$a_R =$ TAB("concrete/RFactor"; a; e=l/t) = 4,219

$I_T = h_R * l * t^3$ = $2{,}213*10^{-3}$ m⁴

$W_T = \frac{l * t^2}{a_R}$ = $8{,}920*10^{-3}$ m³

$T_{Rd,c} = f_{ctd} * W_T * 10^3$ = 10,70 kN/m²

It is only the minimum of reininforcement necessary, if the following proof are sucessfully

$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$ = **7,26 < 1**

The angle of the shearing reinforcement is a = 90°

$V_{Rd,max} = \frac{a_{cw} * b_w * z * f_{cd} * v_1}{\left(\frac{1}{\tan(Q)} + \tan(Q)\right)} * 10^3$ = 846,17 kN

max_V/ $V_{Rd,max}$ = **0,40 < 1**

$f_{ywd} = f_{yd}$ = 434,78 N/mm²

$req\_a_{sw,V} = \frac{V_{Ed}}{z * f_{ywd} * \sin(90) * \left(\frac{1}{\tan(Q)} + \frac{1}{\tan(90)}\right)} * 10$ = 15,07 cm²/m

Minimum shear reinforcement ratio

$r_{w,min} = 0{,}08 * \frac{Ö f_{ck}}{f_{yk}}$ = $0{,}80*10^{-3}$

maximal spacing for links

$s_{t,max} =$ MIN(0,75*d; 0,60) = 0,39 m

$a_{sw,min} = r_{w,min} * 10^4 * b_w *$ SIN(90) = 3,20 cm²/m

$req\_a_{sw,V} =$ MAX($req\_a_{sw,V}$; $a_{sw,min}$) = 15,07 cm²/m

Design for torsion:

$v =$ v_1 = 0,54

$T_{Rd,max} = v * a_{cw} * f_{cd} * 2 * A_k * t_{ef,1} * SIN(Q) * COS(Q)*10^3 =$ 140,43 kN

$T_{Ed} / T_{Rd,max}$ = <u>**0,32 < 1**</u>

tension reinforcement as a result of torsion

$$req\_a_{sl,T} = \frac{T_{Ed} * 10}{2 * A_k * f_{yd} * \tan(Q)} = 3{,}98\ cm^2/m$$

links as a result of torsion

$$req\_a_{sw,T} = \frac{T_{Ed} * 10 * \tan(Q)}{2 * A_k * f_{yd}} = 3{,}98\ cm^2/m$$

$T_{Ed} / T_{Rd,max} + V_{Ed} / V_{Rd,max}$ = <u>**0,68 < 1**</u>

<u>reinforcement links:</u>

$req\_a_{sw} =$ $2 * req\_a_{sw,T} + req\_a_{sw,V}$ = <u>**23,03 cm²/m**</u>

Provide links of size 12 mm (double-shear)

$d_s =$ SEL("reinf/AsArea"; ds;) = 12,00 mm

$a_s =$ SEL("reinf/AsArea";Name;d_s=d_s;a_s ³ req_a_{sw}/2) = Æ 12 / e = 9.5

$prov\_a_{sw}$= 2*TAB("reinf/AsArea"; as; Name=a_s) = 23,80 cm²/m

$req\_a_{sw} / prov\_a_{sw}$ = <u>**0,97 < 1**</u>

<u>torsion reinforcement longitudinal bars:</u>

$req\_A_{Sl} =$ $prov\_A_{S,M} + req\_a_{sl,T} * u_k /3$ = 14,52 cm²

$A_{s,sel} =$ SEL("reinf/As"; Name; As ³ req_A_{Sl}) = 5 Æ 20

$prov\_A_s =$ TAB("reinf/As"; As; Name=$A_{s,sel}$) = 15,71 cm²

provide five bars 20 mm

<u>at the side:</u>

$req\_A_{Sl,T} =$ $req\_a_{sl,T} * u_k /3$ = 2,02 cm²

$A_{s,sel} =$ SEL("reinf/As"; Name; As ³ $req\_A_{Sl,T}$) = 2 Æ 14

$prov\_A_s =$ TAB("reinf/As"; As; Name=$A_{s,sel}$) = 3,08 cm²

provide two bars 14 mm

<u>side of compression</u>

$req\_A_{Sl,T} =$ $req\_a_{sl,T} * u_k /3$ = 2,02 cm²

$A_{s,sel} =$ SEL("reinf/As"; Name; As ³ $req\_A_{Sl,T}$) = 2 Æ 14

$prov\_A_s =$ TAB("reinf/As"; As; Name=$A_{s,sel}$) = 3,08 cm²

provide two bars 14 mm

minimum reinforcement longitudinal bars :

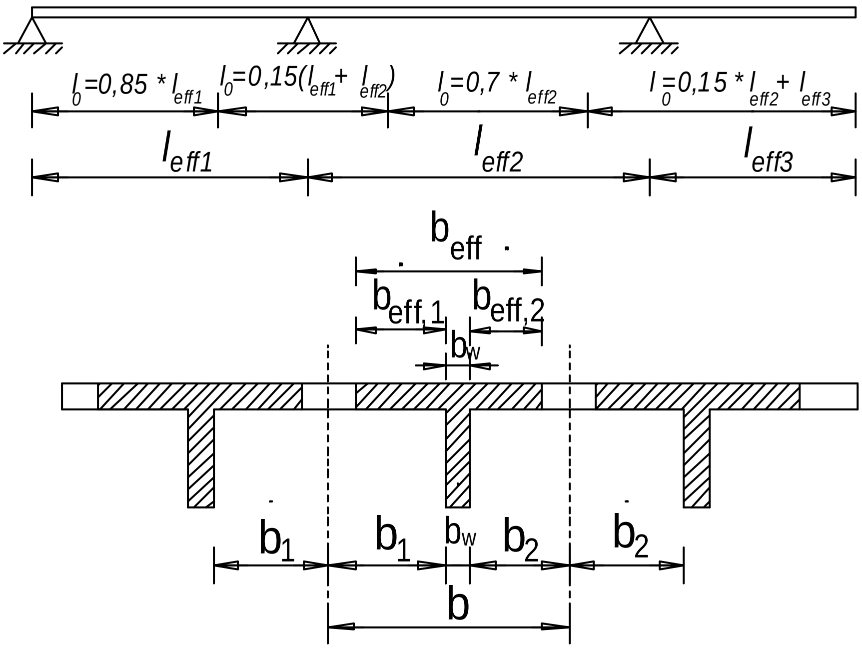
$$A_{Sl,min} = MAX(0{,}26 * \frac{f_{ctm}}{f_{yk}} * b * d * 10^4 ; 0{,}0013 * b * d * 10^4) = 2{,}84\ cm^2$$

$$\frac{A_{Sl,min}}{req\_a_{sl,T} * u_k}$$ = <u>**0,47 £1**</u>

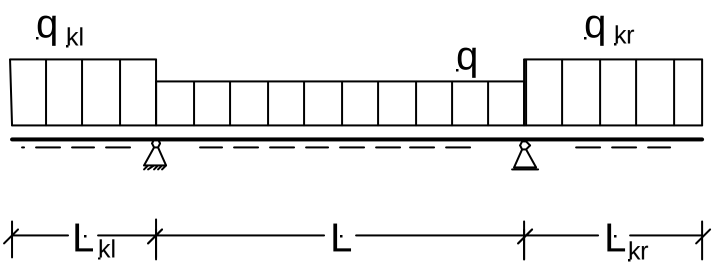
Effective flange width of a single-span beam with cantilevers:

to EN 1992-1-1:2004

Elevation:



Load diagram:



Section properties:

l_n is the clear distance between the faces of supports

length l_n =			6,26 m
bearing width t_1 =			0,30 m
bearing width t_2 =			0,30 m
height h=			0,50 m

a_1 and a_2 are the end distance at either end of the element.

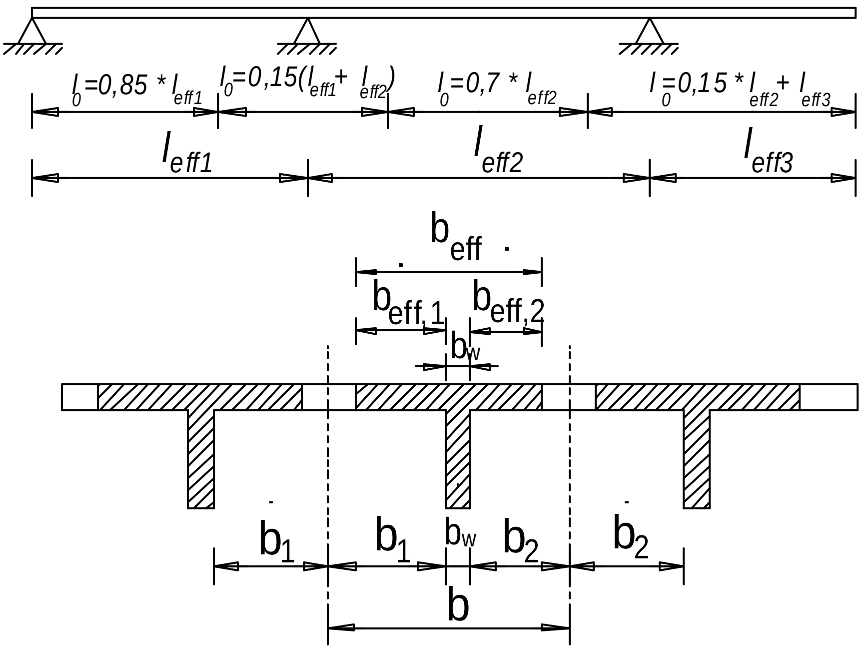
a_1 =	MIN(1/2*h ;1/2*t_1)	=	0,15 m
a_2 =	MIN(1/2*h ;1/2*t_2)	=	0,15 m
Beam width b_w =			0,30 m
half slab width b_1 =			1,75 m
half slab width b_2 =			1,37 m

Single-span beam, top flange:

l_{eff1}=	$l_n+a_1+a_2$	=	6,56 m
l_0=	0,7 * l_{eff1}	=	4,59 m
b =	b_1 + b_2 + b_w	=	3,42 m
b_{eff1} =	MIN(0,2 * b_1 + 0,1 * l_0; 0,2 * l_0; b_1)	=	0,81 m
b_{eff2} =	MIN(0,2 * b_2 + 0,1 * l_0; 0,2 * l_0; b_2)	=	0,73 m
b_{eff} =	MIN(b_{eff1} + b_{eff2} + b_w ; b)	=	1,84 m

Effective flange width at an internal column:
to EN 1992-1-1:2004

Elevation:



Section properties:

l_n is the clear distance between the faces of supports

length l_{n1}=			6,26 m
length l_{n2}=			7,00 m
bearing width t_1 =			0,30 m
bearing width t_2 =			0,30 m
bearing width t_3 =			0,40 m
height h=			0,50 m

a_1 and a_2 are the end distance at either end of the element.

a_1 =	MIN(1/2*h ;1/2*t_1)	=	0,15 m
a_2 =	MIN(1/2*h ;1/2*t_2)	=	0,15 m
a_3 =	MIN(1/2*h ;1/2*t_3)	=	0,20 m
Beam width b_w =			0,30 m
half slab width b_1 =			1,75 m
half slab width b_2 =			1,37 m

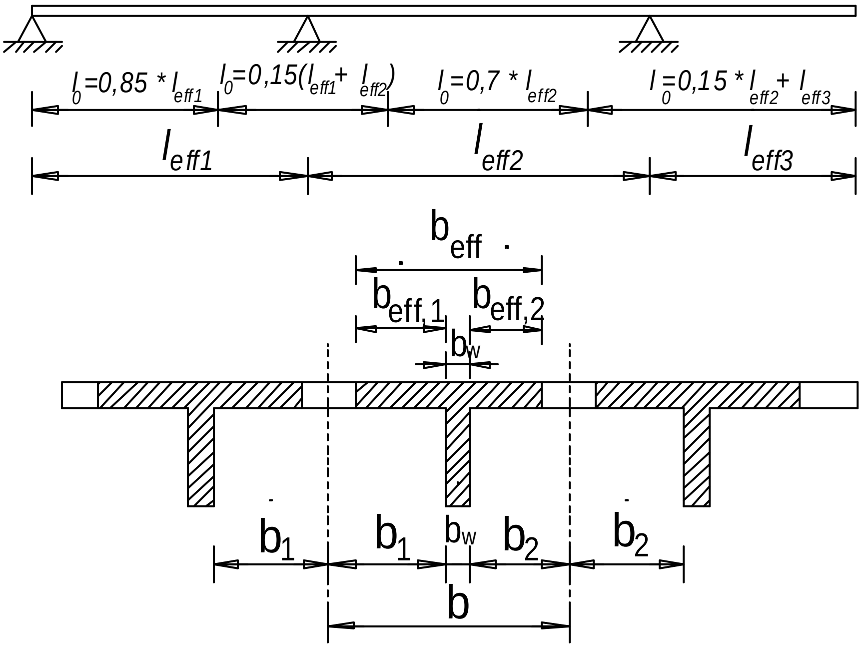
Continuous flanged beam, bottom flange:

l_{eff1}=	$l_{n1}+a_1+a_2$	=	6,56 m
l_{eff2}=	$l_{n2}+a_2+a_3$	=	7,35 m
Innenstütze l_0 =	0,15 * (l_{eff1} + l_{eff2})	=	2,09 m
b =	b_1 + b_2 + b_w	=	3,42 m
b_{eff1} =	MIN(0,2 * b_1 + 0,1 * l_0; 0,2 * l_0; b_1)	=	0,42 m
b_{eff2} =	MIN(0,2 * b_2 + 0,1 * l_0; 0,2 * l_0; b_2)	=	0,42 m
b_{eff} =	MIN(b_{eff1} + b_{eff2} + b_w ; b)	=	1,14 m

Effective flange width of a cantilever beam:

to EN 1992-1-1:2004

Elevation:



Section properties:

The length of the cantilever l_3 should be less than half the adjacent span and the ratio of adjacent spans should lie between 2/3 and 1,5.

length l_{n2}=			6,26 m
length l_{n3}=			2,80 m
bearing width t_2 =			0,30 m
bearing width t_3 =			0,30 m
height h=			0,50 m

a_1 and a_2 are the end distance at either end of the element.

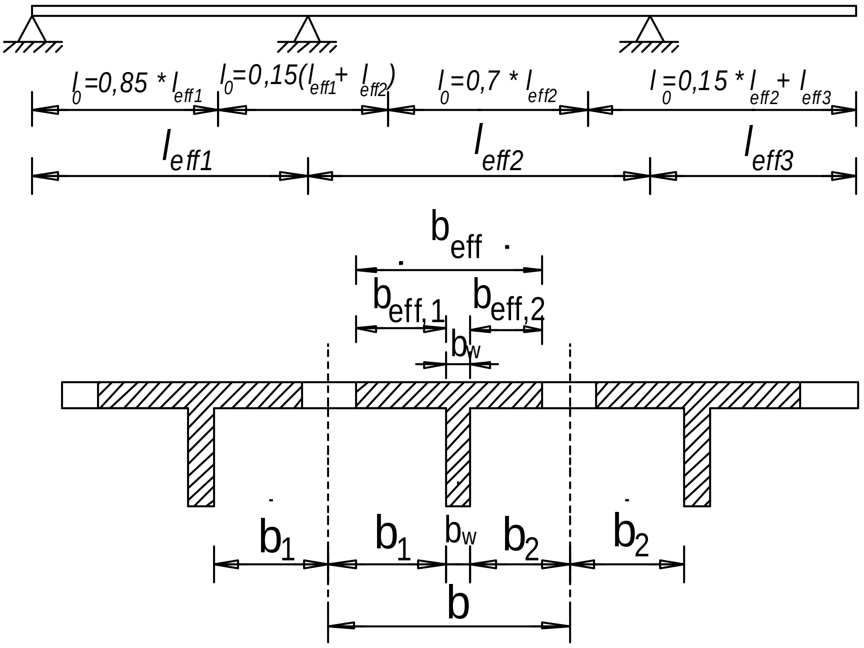
a_2 =	MIN(1/2*h ;1/2*t_2)	=	0,15 m
a_3 =	MIN(1/2*h ;1/2*t_3)	=	0,15 m
Beam width b_w =			0,30 m
half slab width b_1 =			1,75 m
half slab width b_2 =			1,37 m

Cantilever beam, bottom flange:

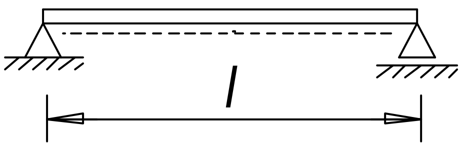
l_{eff2}=	$l_{n2}+a_2+a_3$	=	6,56 m
l_{eff3}=	$l_{n3}+a_3$	=	2,95 m
l_0 =	$0,15*l_{n2} + l_{n3}$	=	3,74 m
b =	$b_1 + b_2 + b_w$	=	3,42 m
b_{eff1} =	MIN(0,2 * b_1 + 0,1 * l_0; 0,2 * l_0; b_1)	=	0,72 m
b_{eff2} =	MIN(0,2 * b_2 + 0,1 * l_0; 0,2 * l_0; b_2)	=	0,65 m
b_{eff} =	MIN(b_{eff1} + b_{eff2} + b_w ; b)	=	1,67 m

Single-span beam with shearing force:
to EN 1992-1-1:2004

Elevation:



Load diagram:



Section properties:

the depth of the stress block lies within the flange ($s \leq h_f$)

l_n is the clear distance between the faces of supports

length l_{n1} =			6,26 m
bearing width t_1=			0,30 m
bearing width t_2=			0,30 m
Beam width b_w =			0,30 m
half slab width b_1 =			1,75 m
half slab width b_2 =			1,37 m
height h =			0,45 m
Slab thickness h_f =			0,15 m
Axis of bending tension reinforcement a =			0,02 m
Assumed bar size d_{s1} =			0,025 m
nom_c =			0,035 m
without alons stress influence σ_{cp} =			0,00

a_1 and a_2 are the end distance at either end of the element.

a_1 =	MIN(1/2*h ;1/2*t_1)	=	0,15 m
a_2 =	MIN(1/2*h ;1/2*t_2)	=	0,15 m

Single-span beam, top flange:

l_{eff1}=	$l_{n1}+a_1+a_2$	=	6,56 m
l_0=	l_{eff1}	=	6,56 m
b =	$b_1 + b_2 + b_w$	=	3,42 m
b_{eff1} =	MIN(0,2 * b_1 + 0,1 * l_0; 0,2 * l_0; b_1)	=	1,01 m
b_{eff2} =	MIN(0,2 * b_2 + 0,1 * l_0; 0,2 * l_0; b_2)	=	0,93 m
b_{eff} =	MIN(b_{eff1} + b_{eff2} + b_w ; b)	=	2,24 m
L =	l_{eff1}	=	6,56 m

Loads:

from dead load:	(b * h_f + b_w * (h - h_f)) * 25	=	15,07 kN/m
From dead floor load:	b * 1,50	=	5,13 kNm
		max q_g =	**20,20 kNm**
from live load:	b * 1,50	=	5,13 kN/m
from partition surcharge:	b * 1,25	=	4,28 kN/m
		max q_q =	**9,41 kN/m**
V_G =	q_g * L / 2	=	66,26 kN
V_Q =	q_q * L / 2	=	30,86 kN
M_G =	q_g * L² / 8	=	108,66 kNm
M_Q =	q_q * L² / 8	=	50,62 kNm

Bending design of retangular cross section:

Materials and stresses:

Concrete =	SEL("concrete/EC"; Name; f_{ck} £ 50)	=	C20/25
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	20,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a cc =			1,00
f_{cd} =	$\frac{f_{ck} * a_{cc}}{1,5}$	=	13,33 N/mm²

Partial safety factors:

g_G =	1,35
g_Q =	1,50

Design Calculation:

M_{Ed} = $g_G * M_G + g_Q * M_Q$ = 222,62 kNm

d = h - nom_c - a - d_{s1} / 2 = 0,383 m

$M_{Ed,s}$ = ABS(M_{Ed}) = 222,62 kNm

$$m_{Ed,s} = \frac{M_{Ed,s} * 10^{-3}}{b_{eff} * d^2 * f_{cd}} = 0{,}051$$

w = TAB("reinf/Ecmy"; w; m=$m_{Ed,s}$) = 0,053

z = TAB("reinf/Ecmy"; z; m=$m_{Ed,s}$) = 0,971

x = TAB("reinf/Ecmy"; x; m=$m_{Ed,s}$) = 0,077

x = x * d = 0,029 m

$$req\_A_s = \frac{w * d * b_{eff} * f_{cd}}{f_{yd}} * 10^4 = 13{,}94 \text{ cm}^2$$

Provide three 25 mm bars

Bar size d_s = SEL("reinf/As"; ds;) = 25 mm

$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s) = 3 Æ 25

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 14,73 cm²

req_A_s / prov_A_s = **0,95 £ 1**

Whole compressed section is in flange:

x / h_f = **0,19 < 1**

Shear Analysis:

max_V = $V_G * g_G + V_Q * g_Q$ = 135,74 kN

Effective design shear force:

V_{Ed} = max_V - ($q_g * g_G + q_q * g_Q$) * (a_1 + d) = 113,68 kN

$$v_1 = 0{,}6 * \left(1 - \frac{f_{ck}}{250}\right) = 0{,}55$$

For flanged beams:

z = d - h_f / 2 = 0,31 m

Limits EC2 for Q : 1 = cot Q = 2,5 (21,8°£ Q £ 45°)

provide cot Q x = 1,50

Consideration of the compressive strength of the diagonal concrete strut and its angle Q

$$Q = \operatorname{atan}\left(\frac{1}{x}\right) = 33{,}69\ °$$

Is a coefficient taking account of the state of the stress in the compression chord.
For non-prestressed structures.

a **cw** = 1,00

The angle of the shearing reinforcement is a = 90°

$$V_{Rd,max} = \frac{a_{cw} * b_w * z * f_{cd} * v_1}{\left(\frac{1}{\tan(Q)} + \tan(Q)\right)} * 10^3 = 314{,}69 \text{ kN}$$

$max\_V / V_{Rd,max}$ = **0,43 < 1**

$f_{ywd} = f_{yd}$ = 434,78 N/mm²

$$req\_a_{sw} = \frac{V_{Ed} \cdot 10^{-3}}{z \cdot f_{ywd} \cdot \frac{1}{\tan(\Theta)}} \cdot 10^4 = 5{,}62 \text{ cm}^2/\text{m}$$

Constructive shearing reinforcement

maximal spacing for links

$s_{t,max}$ = MIN(0,75*d; 0,60) = 0,29 m

$$\rho_{w,min} = 0{,}08 \cdot \frac{\sqrt{f_{ck}}}{f_{yk}} = 0{,}72 \cdot 10^{-3}$$

α = 90,00°

$a_{sw,min} = \rho_{w,min} \cdot 10^4 \cdot b_w \cdot SIN(90)$ = 2,16 cm²/m

req_a_{sw} = MAX(req_a_{sw} ; $a_{sw,min}$) = 5,62 cm²/m

Provide links of size 8 mm (double-shear)

d_s = SEL("reinf/AsArea"; ds;) = 8,00 mm

a_s = SEL("reinf/AsArea";Name;d_s=d_s;$a_s \geq req\_a_{sw}/2$) = Ø 8 / e = 15

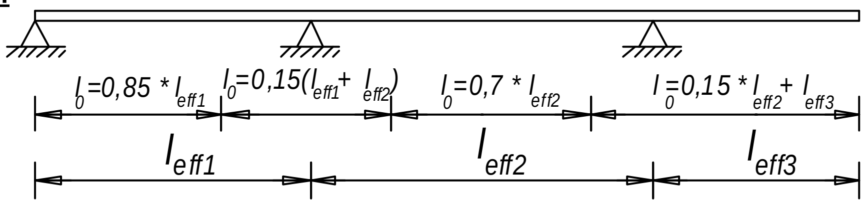
$prov\_a_{sw}$= 2 * TAB("reinf/AsArea"; as; Name=a_s) = 6,70 cm²/m

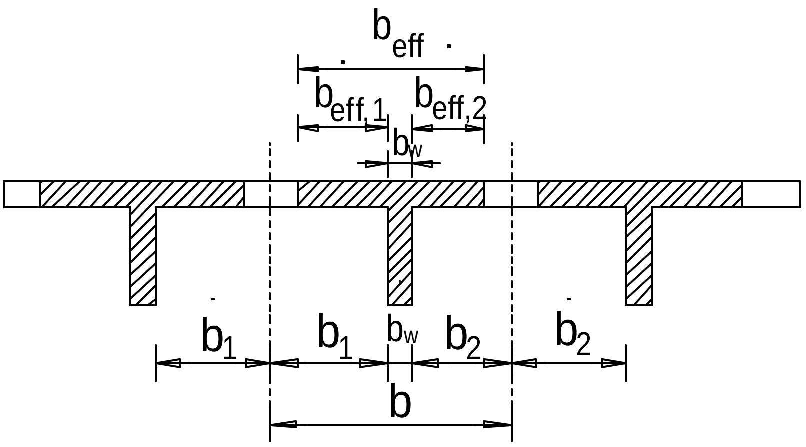
$req\_a_{sw} / prov\_a_{sw}$ = **0,84 < 1**

Design of a single-span flanged beam with the reinforcement omega method:

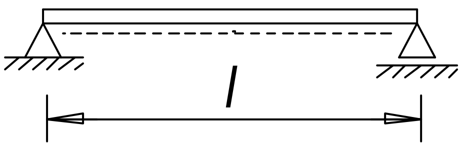
to EN 1992-1-1:2004

Elevation:





Load diagram:



Section properties:

l_n is the clear distance between the faces of supports

length l_{n1} =			6,26 m
bearing width t_1=			0,30 m
bearing width t_2=			0,30 m
Beam width b_w =			0,30 m
half slab width b_1 =			1,75 m
half slab width b_2 =			1,37 m
flanged beam height h =			0,50 m
Flange thickness h_f =			0,15 m
Centroid of tension reinforcement a =			0,02 m
Assumed bar size d_{s1} =			0,025 m
nom_c =			0,035 m

a_1 and a_2 are the end distance at either end of the element.

a_1 =	MIN(1/2*h ;1/2*t_1)	=	0,15 m
a_2 =	MIN(1/2*h ;1/2*t_2)	=	0,15 m

Single-span beam, top flange:

l_{eff1}=	$l_{n1}+a_1+a_2$	=	6,56 m
l_0=	l_{eff1}	=	6,56 m
b =	$b_1 + b_2 + b_w$	=	3,42 m
b_{eff1} =	MIN(0,2 * b_1 + 0,1 * l_0; 0,2 * l_0; b_1)	=	1,01 m
b_{eff2} =	MIN(0,2 * b_2 + 0,1 * l_0; 0,2 * l_0; b_2)	=	0,93 m
b_{eff} =	MIN(b_{eff1} + b_{eff2} + b_w ; b)	=	2,24 m
L =	l_{eff1}	=	6,56 m

Loads:

from dead load:	(b * h_f + b_w * (h - h_f)) * 25	=	15,45 kN/m
From dead floor load:	b * 1,50	=	5,13 kNm
	max q_g	**=**	**20,58 kNm**
from live load:	b * 1,50	=	5,13 kN/m
from partition surcharge:	b * 1,25	=	4,28 kN/m
	max q_q	**=**	**9,41 kN/m**
V_G =	q_g * L / 2	=	67,50 kN
V_Q =	q_q * L / 2	=	30,86 kN
M_G =	q_g * L² / 8	=	110,70 kNm
M_Q =	q_q * L² / 8	=	50,62 kNm

Flexural design of the rectangular section:

Materials and characteristic strengths:

Concrete =	SEL("concrete/EC"; Name; f_{ck} £ 50)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a $_{cc}$ =			1,00
g$_c$ =			1,50
f_{cd} =	$\frac{f_{ck} * a_{cc}}{g_c}$	=	20,00 N/mm²

Partial safety factors:

g$_G$ =	1,35
g$_Q$ =	1,50

Design values and analysis:

M_{Ed} =	$g_G * M_G + g_Q * M_Q$	=	225,38 kNm
d =	h - nom_c - a - d_{s1} / 2	=	0,433 m
$M_{Ed,s}$ =	ABS(M_{Ed})	=	225,38 kNm
$m_{Ed,s}$ =	$\frac{M_{Ed,s} * 10^{-3}}{b_{eff} * d^2 * f_{cd}}$	=	0,027
w =	TAB("reinf/Ecmy"; w; m=$m_{Ed,s}$)	=	0,028
z =	TAB("reinf/Ecmy"; z; m=$m_{Ed,s}$)	=	0,982
x =	TAB("reinf/Ecmy"; x; m=$m_{Ed,s}$)	=	0,052
x =	x * d	=	0,023 m
req_A_s =	$\frac{w * d * b_{eff} * f_{cd}}{f_{yd}} * 10^4$	=	12,49 cm²

Provide five 20 mm bars

Bar size d_s=	SEL("reinf/As"; ds;)	=	20 mm
$A_{s,sel}$ =	SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s)	=	5 Æ 20
prov_A_s =	TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	15,71 cm²

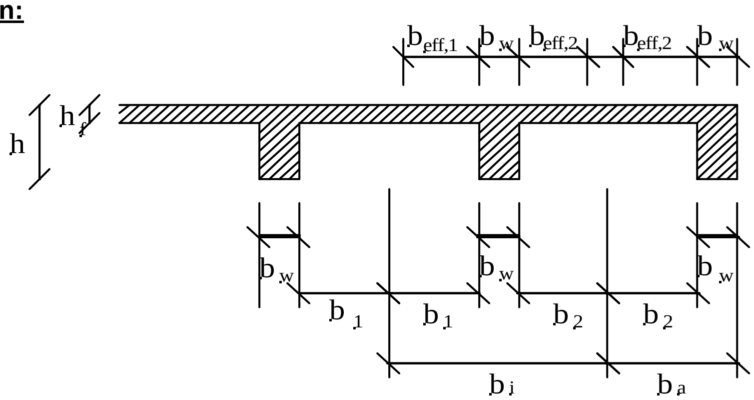
req_A_s / prov_A_s = **0,80 £ 1**

Whole compressed section is in flange:

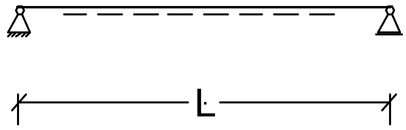
x / h_f = **0,15 < 1**

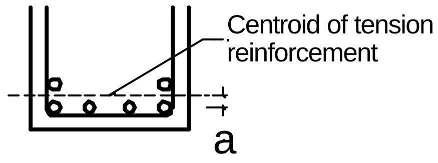
Effective flange width of a single-span beam:
to ENV 1992-1-1 !!!

Elevation:



Load diagram:





Section properties:

Span length l_1 =	6,26 m	
Support width a_1 =	0,30 m	
Support width a_2 =	0,30 m	
Beam width b_w =		0,30 m
half slab width b_1 =	1,75 m	
half slab width b_2 =	1,37 m	
Tee beams height h =		0,45 m
Slab height h_f =		0,15 m
Axis of bending tension reinforcement a =		0,02 m
Assumed bar size d_{s1} =		0,025 m
nom_c =		0,035 m

Single-span beam, top flange:

l_{eff1} =	$l_1 + a_1 / 3 + a_2 / 3$	=	6,46 m
l_0 =	l_{eff1}	=	6,46 m
b_{eff} =	$\mathrm{MIN}(b_w + l_0 / 5;\ b_w + b_1 + b_2)$	=	1,59 m
L =	l_{eff1}	=	6,46
b =	$b_1 + b_2 + b_w$	=	3,42 m

from dead load:	$(b * h_f + b_w * (h - h_f)) * 25$	=	15,07 kN/m
From dead floor load:	b * 1,50	=	5,13 kNm
	max q_g	**=**	**20,20 kNm**
from live load:	b * 1,50	=	5,13 kN/m
from partition surcharge:	b * 1,25	=	4,28 kN/m
	max q_q	**=**	**9,41 kN/m**
V_G =	q_g * L/2	=	65,25 kN
V_Q =	q_q * L/2	=	30,39 kN
M_G =	q_g* L² / 8	=	105,37 kNm
M_Q =	q_q* L² / 8	=	49,09 kNm

Bending design of retangular cross section:

Materials and stresses:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S

Partial safety factors:

g_G = 1,35

g_Q = 1,50

Design Calculation:

M_{sd} =	g_G * M_G + g_Q * M_Q	=	215,88 kNm
d =	h - nom_c - a - d_{s1} / 2	=	0,383 m
z_{s1} =	d - h / 2	=	0,158 m
$M_{sd,s}$ =	ABS(M_{sd})	=	215,88 kNm
k_d =	d * 100 / Ö($M_{sd,s}$ / b_{eff})	=	3,29
k_s =	TAB("reinf/kd"; ks1; Name=Concrete; kd=k_d)	=	2,374
z =	TAB("reinf/kd"; z; Name=Concrete; kd=k_d)	=	0,969
x =	TAB("reinf/kd"; x; Name=Concrete; kd=k_d)	=	0,082

$x =$ $\xi * d$ = 0,031 m
$z =$ $\zeta * d$ = 0,371 m

$req\_A_s =$ $M_{sd,s}/(d*100)*k_s$ = 13,38 cm²

Provide three 25 mm bars

Bar size d_s= SEL("reinf/As"; ds;) = 25 mm
$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s) = 4 Æ 25
$prov\_A_s$ = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 19,63 cm²

req_A_s / $prov\_A_s$ = **0,68 £ 1**

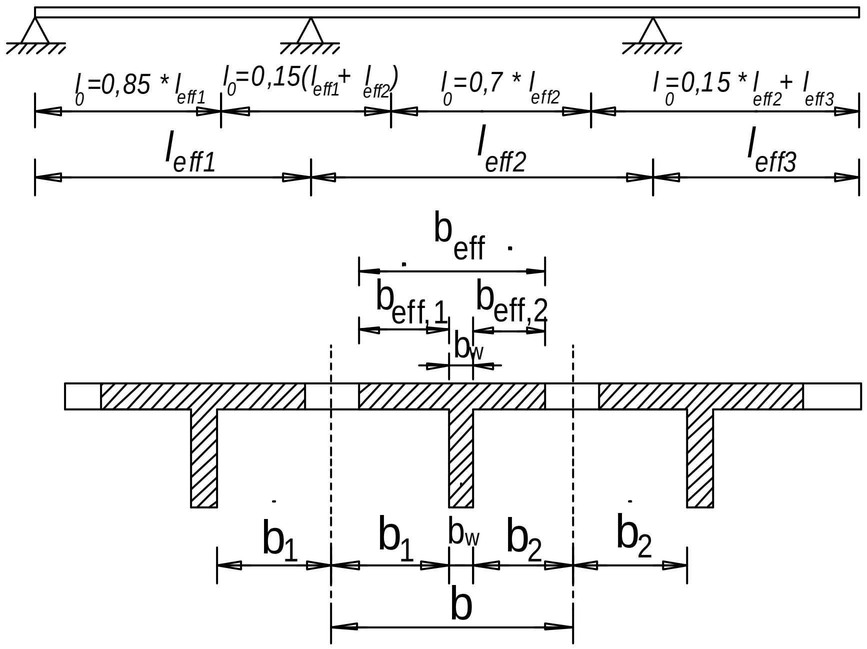
Whole compressed section is in flange:
$x =$ $\xi * d$ = 0,031 m

x / h_f = **0,21 < 1**

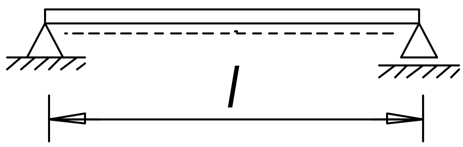
Eff. Flange width Single-span:

to EN 1992-1-1:2004

Elevation:



Load diagram:



Section properties:

l_n is the clear distance between the faces of supports

length l_{n1} =	6,26 m
bearing width t_1=	0,30 m
bearing width t_2=	0,30 m
Beam width b_w =	0,30 m
half slab width b_1 =	1,75 m
half slab width b_2 =	1,37 m
height h =	0,50 m

a_1 and a_2 are the end distance at either end of the element.

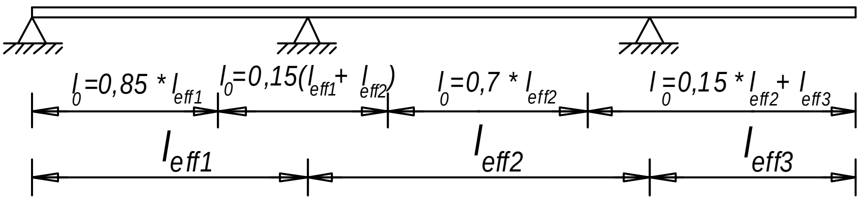
a_1 =	MIN(1/2*h ;1/2*t_1)	=	0,15 m
a_2 =	MIN(1/2*h ;1/2*t_2)	=	0,15 m

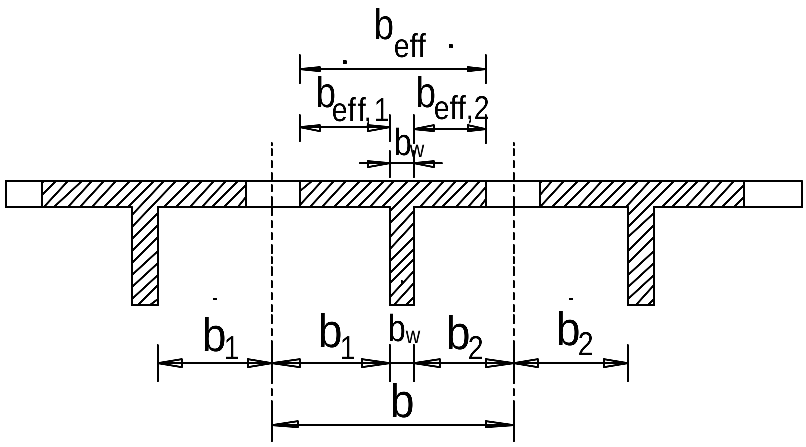
Single-span beam, top flange:

l_{eff1}=	$l_{n1}+a_1+a_2$	=	6,56 m
l_0=	l_{eff1}	=	6,56 m
b =	$b_1 + b_2 + b_w$	=	3,42 m
b_{eff1} =	MIN(0,2 * b_1 + 0,1 * l_0; 0,2 * l_0; b_1)	=	1,01 m
b_{eff2} =	MIN(0,2 * b_2 + 0,1 * l_0; 0,2 * l_0; b_2)	=	0,93 m
b_{eff} =	MIN(b_{eff1} + b_{eff2} + b_w ; b)	=	2,24 m

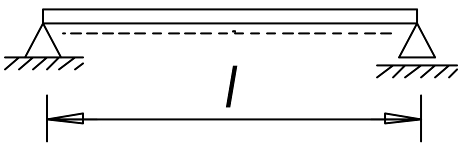
Single-span beam with shearing forces without transverse reinforcement:
to EN 1992-1-1:2004

Elevation:





Load diagram:



Section properties:

the depth of the stress block lies within the flange (s £ h_f)

l_n is the clear distance between the faces of supports

length l_{n1} =			3,00 m
bearing width t_1=			0,30 m
bearing width t_2=			0,30 m
Beam width b_w =			0,30 m
half slab width b_1 =			1,30 m
half slab width b_2 =			1,05 m
height h =			0,30 m
Slab thickness h_f =			0,15 m
Axis of bending tension reinforcement a =			0,02 m
Assumed bar size d_{s1} =			0,025 m
nom_c =			0,035 m

a_1 and a_2 are the end distance at either end of the element.

a_1 =	MIN(1/2*h ;1/2*t_1)	=	0,15 m
a_2 =	MIN(1/2*h ;1/2*t_2)	=	0,15 m

Single-span beam, top flange:

l_{eff1}=	$l_{n1}+a_1+a_2$	=	3,30 m
l_0=	l_{eff1}	=	3,30 m
b =	$b_1 + b_2 + b_w$	=	2,65 m
b_{eff1} =	MIN(0,2 * b_1 + 0,1 * l_0; 0,2 * l_0; b_1)	=	0,59 m
b_{eff2} =	MIN(0,2 * b_2 + 0,1 * l_0; 0,2 * l_0; b_2)	=	0,54 m
b_{eff} =	MIN(b_{eff1} + b_{eff2} + b_w ; b)	=	1,43 m
L =	l_{eff1}	=	3,30 m

Loads:

from dead load:	(b * h_f + b_w * (h - h_f)) * 25	=	11,06 kN/m
From dead floor load:	b * 1,50	=	3,98 kNm
		max q_g =	**15,04 kNm**
from live load:	b * 1,50	=	3,98 kN/m
from partition surcharge:	b * 1,25	=	3,31 kN/m
		max q_q =	**7,29 kN/m**

V_G =	q_g * L / 2	=	24,82 kN
V_Q =	q_q * L / 2	=	12,03 kN
M_G =	q_g * L² / 8	=	20,47 kNm
M_Q =	q_q * L² / 8	=	9,92 kNm

Bending design of retangular cross section:

Materials and stresses:

Concrete =	SEL("concrete/EC"; Name; f_{ck} £ 50)	=	C20/25
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	20,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a **cc** =			1,00
f_{cd} =	$\frac{f_{ck} * a_{cc}}{1,5}$	=	13,33 N/mm²

Partial safety factors:

g_G =	1,35
g_Q =	1,50
g_c =	1,50

Design Calculation:

M_{Ed} = g_G * M_G + g_Q * M_Q = 42,51 kNm

d = h - nom_c - a - d_{s1} / 2 = 0,233 m

$M_{Ed,s}$ = ABS(M_{Ed}) = 42,51 kNm

$m_{Ed,s} = \dfrac{M_{Ed,s} * 10^{-3}}{b_{eff} * d^{2} * f_{cd}}$ = 0,041

w = TAB("reinf/Ecmy"; w; m=$m_{Ed,s}$) = 0,042

z = TAB("reinf/Ecmy"; z; m=$m_{Ed,s}$) = 0,976

x = TAB("reinf/Ecmy"; x; m=$m_{Ed,s}$) = 0,067

x = x * d = 0,016 m

$req\_A_s = \dfrac{w * d * b_{eff} * f_{cd}}{f_{yd}} * 10^{4}$ = 4,29 cm²

Provide three 25 mm bars

Bar size d_s= SEL("reinf/As"; ds;) = 25 mm

$A_{s,sel}$ = SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s) = 3 Æ 25

prov_A_s = TAB("reinf/As"; As; Name=$A_{s,sel}$) = 14,73 cm²

req_A_s / prov_A_s = **0,29 £ 1**

Whole compressed section is in flange:

x / h_f = **0,11 < 1**

Shear Analysis:

max_V = V_G * g_G + V_Q * g_Q = 51,55 kN

Effective shearing force:

V_{Ed} = max_V - (q_g * g_G + q_q * g_Q) * (a_1 + d) = 39,59 kN

k = MIN(1 + Ö$\overline{\dfrac{200}{d * 10^{3}}}$; 2) = 1,93

r_1 = MIN($\dfrac{prov\_A_s}{b_w * d * 10^{4}}$; 0,02) = 0,020

k_1 = 0,15

$C_{Rd,c}$ = 0,18/g_c = 0,12

Acceptance:

Concrete compressive stress at the centroidal axis due to axial loading and/or presstressing s_{cp}=0;

s_{cp} = 0,00 N/mm²

$V_{Rd,c}$ = $(C_{Rd,c} * k * {}^{3}Ö\overline{100 * r_1 * f_{ck}} + k_1 * s_{cp}) * b_w * d * 10^{3}$ = 55,37 kN

u_{min} = 0,035 * Ö$\overline{k^{3}}$ * Ö$\overline{f_{ck}}$ = 0,42 N/mm²

$V_{Rd,c,min}$ = (u_{min} + k_1 * s_{cp}) * b_w * d*10³ = 29,36 kN

$V_{Rd,c}$ = MAX($V_{Rd,c}$; $V_{Rd,c,min}$) = 55,37 kN

$V_{Ed} / V_{Rd,c}$ = **$0,72 \leq 1$**

Shear reinforcement is not necessary, if proof is $\leq$ 1.

Constructive shearing reinforcement

$\rho_{w,min} = 0,08 * \frac{\sqrt{f_{ck}}}{f_{yk}}$ = $0,72*10^{-3}$

$s_{t,max}$ = MIN(0,75*d; 0,60) = 0,17 m

α = 90,00°

$a_{sw,min}$ = $\rho_{w,min}$ * 10^4 * b_w * SIN(90) = 2,16 cm²/m

Provide links of size 8 mm (double-shear)

d_s = SEL("reinf/AsArea"; ds;) = 8,00 mm

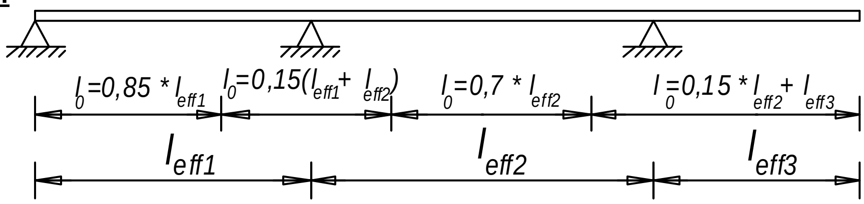
a_s = SEL("reinf/AsArea";Name;d_s=d_s;$a_s \geq a_{sw,min}$/2) $\varnothing$ 8 / e = 17

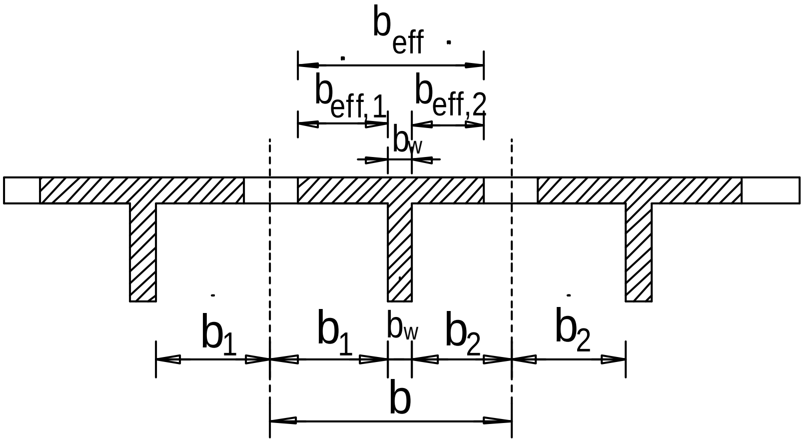
prov_a_{sw}= 2 * TAB("reinf/AsArea"; as; Name=a_s) = 5,92 cm²/m

$a_{sw,min}$ / prov_a_{sw} = **0,36 < 1**

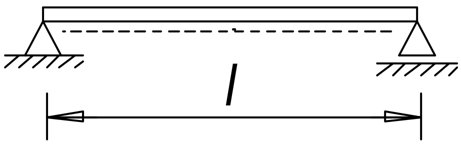
Design of a single-span flanged beam with the reinforcement ratio method:
to EN 1992-1-1:2004

Elevation:





Load diagram:



Section properties:

the depth of the stress block lies within the flange (s £ h_f)

l_n is the clear distance between the faces of supports

length l_{n1} =	6,26 m
bearing width t_1=	0,30 m
bearing width t_2=	0,30 m
Beam width b_w =	0,30 m
half slab width b_1 =	1,75 m
half slab width b_2 =	1,37 m
flanged beam height h =	0,50 m
Flange thickness h_f =	0,15 m
Centroid of tension reinforcement a =	0,02 m
Assumed bar size d_{s1} =	0,025 m
nom_c =	0,035 m

a_1 and a_2 are the end distance at either end of the element.

a_1 =	MIN(1/2*h ;1/2*t_1)	=	0,15 m
a_2 =	MIN(1/2*h ;1/2*t_2)	=	0,15 m

Single-span beam, top flange:

l_{eff1}=	$l_{n1}+a_1+a_2$	=	6,56 m
l_0=	l_{eff1}	=	6,56 m
b =	$b_1 + b_2 + b_w$	=	3,42 m
b_{eff1} =	MIN(0,2 * b_1 + 0,1 * l_0; 0,2 * l_0; b_1)	=	1,01 m
b_{eff2} =	MIN(0,2 * b_2 + 0,1 * l_0; 0,2 * l_0; b_2)	=	0,93 m
b_{eff} =	MIN(b_{eff1} + b_{eff2} + b_w ; b)	=	2,24 m
L =	l_{eff1}	=	6,56 m

Loads:

from dead load:	(b * h_f + b_w * (h - h_f)) * 25	=	15,45 kN/m
From dead floor load:	b * 1,50	=	5,13 kNm
		max q_g =	**20,58 kNm**
from live load:	b * 1,50	=	5,13 kN/m
from partition surcharge:	b * 1,25	=	4,28 kN/m
		max q_q =	**9,41 kN/m**

V_G =	q_g * L / 2	=	67,50 kN
V_Q =	q_q * L / 2	=	30,86 kN
M_G =	q_g * L² / 8	=	110,70 kNm
M_Q =	q_q * L² / 8	=	50,62 kNm

Flexural design of the rectangular section:

Materials and characteristic strengths:

Concrete =	SEL("concrete/EC"; Name; f_{ck} £ 50)	=	C30/37
Steel =	SEL("reinf/Steel"; Name;)	=	500 S
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²
f_{yk} =	TAB("reinf/Steel"; bs; Name=Steel)	=	500 MN/m²
f_{yd}=	f_{yk} / 1,15	=	434,78 N/mm²
a_{cc} =			1,00
g_c =			1,50
f_{cd} =	$\frac{f_{ck} * a_{cc}}{g_c}$	=	20,00 N/mm²

Partial safety factors:

g_G =	1,35
g_Q =	1,50

Design values and analysis:

M_{Ed} =	$g_G * M_G + g_Q * M_Q$	=	225,38 kNm
d =	$h - nom\_c - a - d_{s1} / 2$	=	0,433 m
$M_{Ed,s}$ =	$ABS(M_{Ed})$	=	225,38 kNm
$m_{Ed,s}$ =	$\frac{M_{Ed,s} * 10^{-3}}{b_{eff} * d^2 * f_{cd}}$	=	0,027
z =	TAB("reinf/Ecmy"; z; $m=m_{Ed,s}$)	=	0,982
x =	TAB("reinf/Ecmy"; x; $m=m_{Ed,s}$)	=	0,052
x =	x * d	=	0,023 m
z =	z * d	=	0,425 m
req_A_s =	$\frac{1}{f_{yd}} * \left(\frac{M_{Ed,s}}{d - \frac{h_f}{2}} \right) *10$	=	14,48 cm²

Provide five 20 mm bars

Bar size d_s=	SEL("reinf/As"; ds;)	=	20 mm
$A_{s,sel}$ =	SEL("reinf/As"; Name; d_s = d_s; As ³ req_A_s)	=	5 Æ 20
$prov\_A_s$ =	TAB("reinf/As"; As; Name=$A_{s,sel}$)	=	15,71 cm²

$req\_A_s / prov\_A_s$ = **0,92 £ 1**

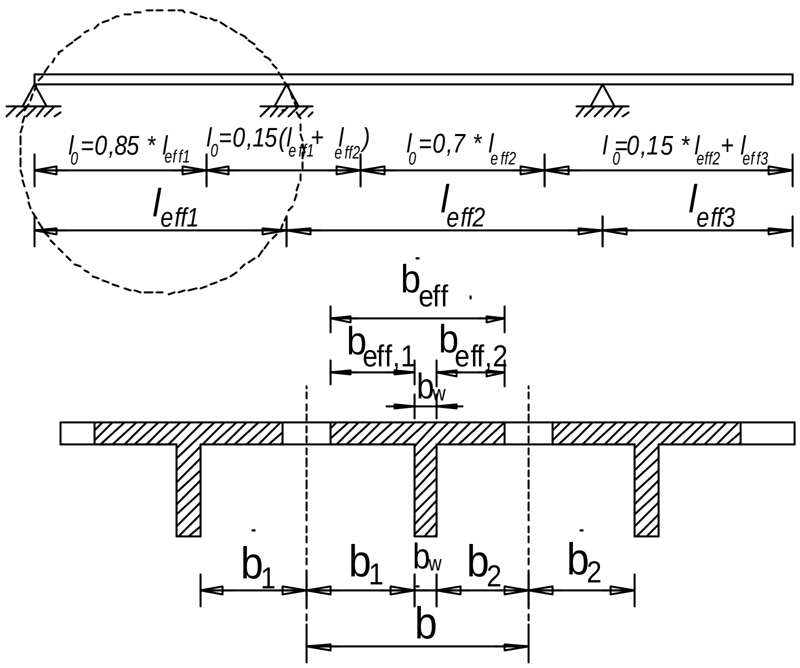
Whole compressed section is in flange:

x / h_f = **0,15 < 1**

Effective flange width of a single-span with a cantilever beam or endfield:

to EN 1992-1-1:2004

Elevation:



Section properties:

l_n is the clear distance between the faces of supports

length l_{n1}=			6,26 m
bearing width t_1 =			0,30 m
bearing width t_2 =			0,30 m
height h=			0,50 m

a_1 and a_2 are the end distance at either end of the element.

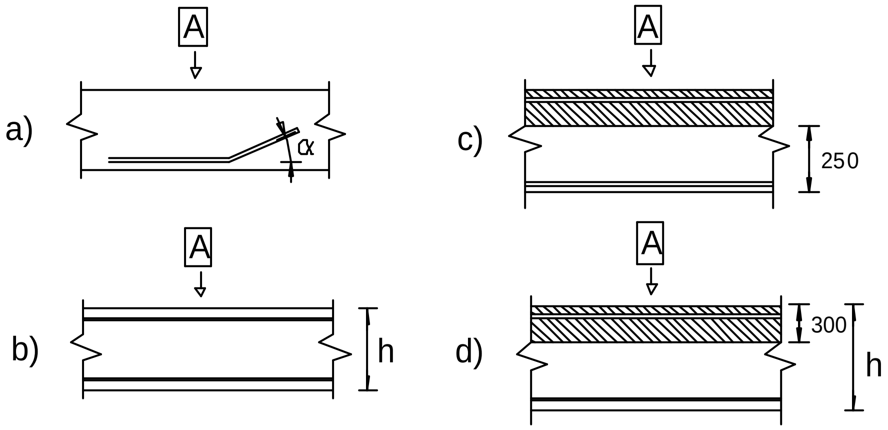
a_1 =	MIN(1/2*h ;1/2*t_1)	=	0,15 m
a_2 =	MIN(1/2*h ;1/2*t_2)	=	0,15 m
Beam width b_w =			0,30 m
half slab width b_1 =			1,75 m
half slab width b_2 =			1,37 m

Continuous flanged beam, bottom flange:

l_{eff1}=	$l_{n1}+a_1+a_2$	=	6,56 m
Innenstütze l_0 =	0,85 * l_{eff1}	=	5,58 m
b =	$b_1 + b_2 + b_w$	=	3,42 m
b_{eff1} =	MIN(0,2 * b_1 + 0,1 * l_0; 0,2 * l_0; b_1)	=	0,91 m
b_{eff2} =	MIN(0,2 * b_2 + 0,1 * l_0; 0,2 * l_0; b_2)	=	0,83 m
b_{eff} =	MIN(b_{eff1} + b_{eff2} + b_w ; b)	=	2,04 m

General anchorage-bond length:

to EN 1992-1-1:2004



a) 45° £ a £ 90°

b) h £ 250 mm

c) h > 250 mm

d) h > 600 mm

A - direction of concreting

a) and b) Good bond conditions for all bars
c) and d) Good bond conditions in unhatched zone. Poor bond conditions in hatched zone.

Section properties:

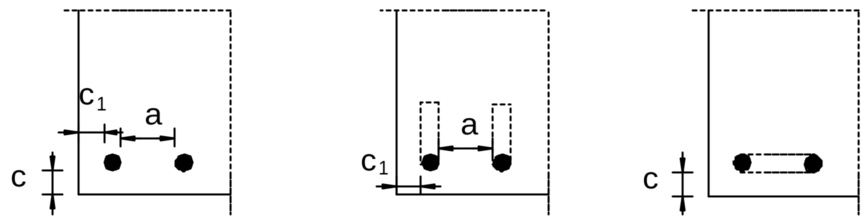
Transformed Value of moment in the level of tension reinforcement. $M_{Ed,s}$ =ABS(M_{Ed})- N_{Ed}

M_{Eds} =	0,308 MN
N_{Ed} =	0,100 MN
z =	0,552 m
prov_A_{s1} =	25,00 cm²

available condition AC= SEL("reinf/aTyps";AC;) = poor

When bond conditions are poor then the specified ultimate bond stress should be reduced by a factor of 0,7

factor η_1 = IF(AC="good";1,0;0,7) = 0,70

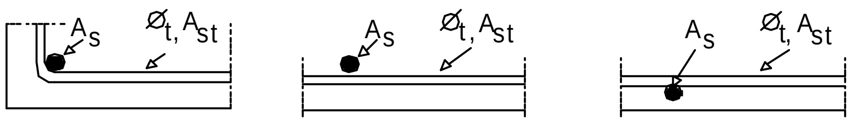


a) Straight bars   b) Bent or hooked bars   c) Looped bars

c_d= min(a/2, c, c1) c_d = min(a/2, c1) c_d = c

concrete cover coefficent c_d = 30 mm

Values of K for beams and slabs



K= 0,1 K= 0,05 K= 0

Value of K = 0,10
transverse pressure p= 0,00 N/mm²
Value of a (1 to 5)
Type of anchorange
VA = SEL("reinf/aTyps";Typ;) = other than straight
f_1 = TAB("reinf/aTyps";Val;Typ=VA) = 2
Construct CS = SEL("reinf/aTyps"; CS;) = beams

Material

Bar size d_s= SEL("reinf/AsArea"; ds;) = 20 mm
Concrete = SEL("reinf/al"; Name;) = C35/45

Area of a single anchored bar with maximum bar diameter
Area A_s= TAB("Bewehrung/As"; As ;n=1;ds=ds) = 3,14 cm²
Bar Size transverse reinforcement along the design anchorage length
Æ d_t = SEL("reinf/AsArea"; ds;) = 10,00 mm
quantity of transverse reinforcement along the design anchorage length
n_t = 4
the cross-sectional area of the transverse reinforcement along the design anchorage length
S A_{st} = TAB("reinf/As"; As ;n=n_t ;ds=d_t) = 3,14 cm²
$f_{ctk\_0,05}$ = TAB("concrete/EC";fctk05; Name=Concrete) = 2,20 N/mm²
a_{ct} = 1,00
g_C = 1,50
f_{yk} = 500,0 N/mm²
g_S = 1,15
E-Modul E_s = 200000 N/mm²

Berechnung

f_{ctd} = $a_{ct} * \frac{f_{ctk\_0,05}}{g_C}$ = 1,47 N/mm²

The design value of the ultimate bond stress is also dependent on the bar size. For all sizes Æ greater than 32 mm the bond stress should additionally be multiplied by a faktor.
h_2 = IF(d_s £ 32; 1,0; (132-d_s)/100) = 1,00
Design values of bond stresses f_{bd}
f_{bd} = 2,25 * h_1* h_2 * f_{ctd} = 2,32 N/mm²

Value of stress where the anchorange of the bar be calculated.

s_{sd} = $\frac{1}{prov\_A_{s1} * 10^{-4}} * \left(\frac{M_{Eds}}{z} + N_{Ed}\right)$ = 263 N/mm²

basic reqiured anchorage length to prevent pull out

$l_{b,rqd}$ = $\frac{d_s}{4} * \frac{s_{sd}}{f_{bd}}$ = 567 mm

the cross-sectional area of the minimum transverse reinforcement

$SA_{st,min}$ = IF(CS ="beams"; 0,25*A_s ; 0) = 0,79 cm²

<u>anchorage-bond length for tension</u>

a_1 for the effect of: The shape of the bars

a_1 = IF(f_1 =1;1;IF(c_d>3*d_s ;0,7;1,0)) = 1,00

a_2 = for the effect of: Concrete cover to the reinforcement

$a_{2,1}$ = $1 - 0,15 * \frac{(c_d - d_s)}{d_s}$ = 0,93

$a_{2,1}$ = IF($a_{2,1}$ < 0,7; 0,7;IF($a_{2,1}$ > 1,0; 1,0; $a_{2,1}$)) = 0,93

$a_{2,2}$ = $1 - 0,15 * \frac{(c_d - d_s * 3)}{d_s}$ = 1,23

$a_{2,2}$ = IF($a_{2,2}$ < 0,7; 0,7;IF($a_{2,2}$ > 1,0; 1,0; $a_{2,2}$)) = 1,00

a_2 = IF(f_1 =1; $a_{2,1}$; $a_{2,2}$) = 1,00

a_2 = IF(f_1 =1; $a_{2,1}$; $a_{2,2}$) = 1,00

a_3 = for the effect of: Confinement of transverse reinforcement not welded to the main reinforcement

l = (SA_{st} - $SA_{st,min}$)/A_s = 0,75

a_3 = 1 - K * l = 0,93

a_3 = IF(a_3 < 0,7; 0,7;WENN(a_3 > 1,0; 1,0; a_3)) = 0,93

a_4 = for the effect of: Confinement of transverse reinforcement welded to the main reinforcement

a_4 = 0,70

a_5 = for the effect of: Confinement by transverse pressure

a_5 = 1-0,04*p = 1,00

a_5 = IF(a_5 < 0,7; 0,7; IF(a_5 > 1,0; 1,0; a_5)) = 1,00

Design value of bond stresses l_{bd} with transverse reinforcement but not welded transverse reinforcement to the main reinforcement.

l_{bd} = $a_1 * a_2 * a_3 * a_5 * l_{b,rqd}$ = 527,31 mm

Design value of bond stresses l_{bd} with welded transverse reinforcement to the main reinforcement.

l_{bd} = $a_1 * a_2 * a_4 * a_5 * l_{b,rqd}$ = 396,90 mm

Design value of bond stresses l_{bd} without transverse reinforcement and not welded transverse reinforcement to the main reinforcement.

l_{bd} = $a_1 * a_2 * a_5 * l_{b,rqd}$ = 567,00 mm

Design value of bond stresses l_{bd} with all factors of a

l_{bd} = $a_1 * a_2 * a_3 * a_4 * a_5 * l_{b,rqd}$ = 369,12 mm

<u>This minimum design length must not be less than:</u>
<u>for tension bars:</u>

$l_{b,min,t}$ = MAX(0,3*$l_{b,rqd}$; 10 * d_s ; 100) = 200,00 mm

Anchorage-bond length for compression

Design value of bond stresses l_{bd} with welded transverse reinforcement to the main reinforcement.

l_{bd} = a_4 * $l_{b,rqd}$ = 396,90 mm

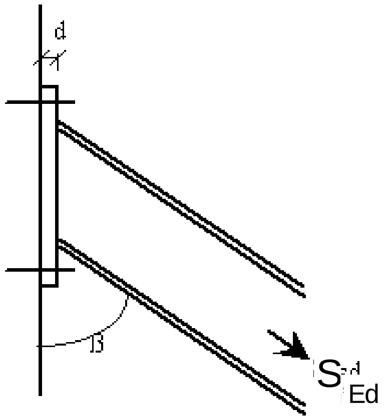
Design value of bond stresses l_{bd} not welded transverse reinforcement to the main reinforcement.

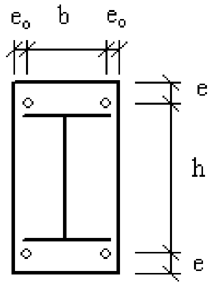
l_{bd} = $l_{b,rqd}$ = 567,00 mm

This minimum design length must not be less than:
for compression bars:

$l_{b,min,d}$ = MAX(0,6*$l_{b,rqd}$; 10 * d_s ; 100) = 340,20 mm

Bolted angilar connection:





Dimensions of connection:

Plate thickness d =	15,00 mm
Bolt spacing e =	35,00 mm
Spacing of bolts e_o =	30,00 mm
Spacing of bolts b =	70,00 mm
Spacing of bolts h =	220,00 mm
Angle of tension force b =	45,00 °

Bolts:

Bolt =	SEL("steel/bolt"; BS;)	=	M 20
SC =	SEL("steel/bolt"; SC;)	=	8.8
$f_{u,b}$ =	TAB("steel/bolt"; fubk; SC=SC)	=	800,00 N/mm²
Hole diameter d_l=	TAB("steel/bolt"; d; BS=Bolt)	=	20,00 mm
Shaft diameter d_{ll}=	d_l+2	=	22,00 mm
Cross-section areas A_s =	TAB("steel/bolt"; Asp; BS=Bolt)	=	2,45 cm²

Plate:

steel =	SEL("steel/EC"; Name;)	=	Fe 510
f_y =	TAB("steel/EC"; fy; Name=steel)	=	355,00 N/mm²
f_u =	TAB("steel/EC"; fu; Name=steel)	=	510,00 N/mm²
g_{M0}=	1,10		
g_{Mb}=	1,25		

Loads:

S_{Ed} =			424,26 kN
N_{Ed} =	COS(b) * S_{Ed}	=	300,00 kN
V_{Ed} =	SIN(b) * S_{Ed}	=	300,00 kN

Force per bolt:

$$F_{t,Ed} = \frac{N_{Ed}}{4} = 75,00 \text{ kN}$$

$$F_{v,Ed} = \frac{V_{Ed}}{4} = 75,00 \text{ kN}$$

Limit tension force of bolts:

$$F_{t,Rd} = 0{,}9 * \frac{f_{u,b} * A_s}{g_{Mb} * 10} = 141{,}12 \text{ kN}$$

$$\frac{F_{t,Ed}}{F_{t,Rd}} = \underline{0{,}53 < 1}$$

Check limit shear force for bolts:

$$F_{v,Rd} = 0{,}6 * \frac{f_{u,b} * p * d_l^{\,2}}{4000 * g_{Mb}} = 120{,}64 \text{ kN}$$

$$\frac{F_{v,Ed}}{F_{v,Rd}} = \underline{0{,}62 < 1}$$

Check combined:

$$\frac{F_{t,Ed}}{1{,}4 * F_{t,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} = \underline{1{,}00 \text{ £ } 1}$$

Analysis of bearing strength:

as in 6.5.1.3:	e_o / d_{ll}	=	1,36 > 1
as in 6.5.1.3:	e_o / d_{ll}	=	1,36 < 1,5
as in 6.5.1.2(1):	e / d_{ll}	=	1,59 > 1,2
as in 6.5.1.2(3):	b / d_{ll}	=	3,18 > 3

$$a = MIN\left(\frac{e}{3 * d_{ll}}; \frac{b}{3 * d_{ll}} - 0{,}25; \frac{f_{u,b}}{f_u}; 1\right) = 0{,}53$$

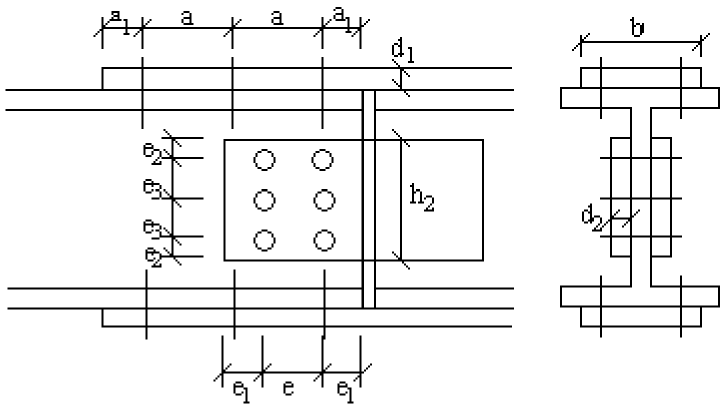
$$\text{Tab.6.5.3 } F_{b1,Rd} = 2{,}5 * a * d_l * d * \frac{f_u}{g_{Mb} * 10^3} = 162{,}18 \text{ kN}$$

$$\text{6.5.5.(10) } F_{b2,Rd} = \frac{2}{3} * F_{b1,Rd} = 108{,}12 \text{ kN}$$

$$F_{b,Rd} = F_{b2,Rd} + (F_{b1,Rd} - F_{b2,Rd}) * \frac{\frac{e_o}{d_l} - 1{,}2}{0{,}3} = 162{,}18 \text{ kN}$$

$$\frac{F_{v,Ed}}{F_{b,Rd}} = \underline{0{,}46 < 1}$$

Bolted connection subject to bending



Dimensions of connection:

a =	80,00 mm		
a_1 =	45,00 mm		
e =	80,00 mm		
e_1 =	45,00 mm		
e_2 =	50,00 mm		
e_3 =	90,00 mm		
d_1 =	20,00 mm		
d_2 =	8,00 mm		
b_1 =	160,00 mm		
h_2 =	$2 * (e_2 + e_3)$	=	280,00 mm

Flange-Bolts:

$Bolt_f$ =	SEL("steel/bolt"; BS;)	=	M 20
SC1 =	SEL("steel/bolt"; SC;)	=	4.6
$f_{u,bo}$ =	TAB("steel/bolt"; fubk; SC=SC1)	=	400,00 N/mm²
Hole diameter d_{lo} =	TAB("steel/bolt"; d; BS=$Bolt_f$)	=	20,00 mm
Shaft diameter d_{l1o} =	d_{lo} + 2	=	22,00 mm
Number of rows n_o =			3

Steg-Bolts:

$Bolt_S$ =	SEL("steel/bolt"; BS;)	=	M 16
SC2 =	SEL("steel/bolt"; SC;)	=	4.6
$f_{u,bs}$ =	TAB("steel/bolt"; fubk; SC=SC2)	=	400,00 N/mm²
Hole diameter d_{ls} =	TAB("steel/bolt"; d; BS=$Bolt_S$)	=	16,00 mm
Shaft diameter d_{l1s} =	d_{ls} + 2	=	18,00 mm
Number of rows n_s =			3
Number of columns m_s =			2

Profil:

Profil Typ =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil =	SEL("steel/"Typ; Name;)	=	IPE 240
Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	240,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	6,20 mm
Flange thickness t =	TAB("steel/"Typ; t; Name=Profil)	=	9,80 mm
Cross-sectional area A =	TAB("steel/"Typ; A; Name=Profil)	=	39,10 cm²
Moment of resistance W_y =	TAB("steel/"Typ; Wy; Name=Profil)	=	324,00 cm²
Moment of resistance W_{pl} =	1,14 * W_y	=	369,36 cm³

Material and Partial safety factors:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
γ_{M0} =	1,10		
γ_{M2} =	1,25		

Loads:

M_{Ed} = 157,95 kNm

V_{Ed} = 60,75 kN

Check beam-web strength without holes:

$$M_{c,Rd} = \frac{W_{pl} * f_y}{\gamma_{M0} * 10^3} = 78,91 \text{ kNm}$$

$$\frac{M_{Ed}}{M_{c,Rd}} = \underline{\mathbf{2,00 < 1}}$$

$$A_v = 1,04 * \frac{h * s}{100} = 15,48 \text{ cm}^2$$

$$V_{pl,Rd} = A_v * \frac{f_y}{\sqrt{3} * \gamma_{M0} * 10} = 190,93 \text{ kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \underline{\mathbf{0,32 < 1}}$$

Check beam-web strength with holes:

$$A_z = 0,5 * A = 19,55 \text{ cm}^2$$

$$A_{z,net} = A_z - \frac{2 * d_{l1o} * t + \frac{n_s - 1}{2} * d_{l1s} * s}{100} = 14,12 \text{ cm}^2$$

$$\frac{\frac{f_y}{f_u} * \frac{\gamma_{M2}}{\gamma_{M0}}}{0,9 * \frac{A_{z,net}}{A_z}} = \underline{\mathbf{1,14 \le 1}}$$

$$W_{pl,net} = W_{pl} - 2 * d_{l1o} * t * \frac{h - t}{2000} = 319{,}73\ cm^3$$

$$M_{c,Rd,C} = \frac{W_{pl,net} * f_y}{g_{M0} * 10^3} = 68{,}31\ kNm$$

$$\frac{M_{Ed}}{M_{c,Rd,C}} = \underline{2{,}31 < 1}$$

$$A_{v,net} = A_v - n_s * d_{l1s} * \frac{s}{100} = 12{,}13\ cm^2$$

$$\frac{f_y}{f_u} * \frac{A_v}{A_{v,net}} = \underline{0{,}83 < 1}$$

⇒ Area of holes will be neglected.

Analysis of bolts in flange:

$$I_w = \frac{2 * d_2 * h_2^3}{12 * 10^4} = 2926{,}93\ cm^4$$

$$I_f = 2 * b_1 * d_1 * \frac{\left(\frac{h + d_1}{2}\right)^2}{10^4} = 10816{,}00\ cm^4$$

$$M_w = M_{Ed} * \frac{I_w}{I_w + I_f} = 33{,}64\ kNm$$

$$M_f = M_{Ed} * \frac{I_f}{I_w + I_f} = 124{,}31\ kNm$$

Effective tension force in flange:

$$N_{Ed} = \frac{M_f}{h + d_1} * 10^3 = 478{,}12\ kN$$

Limit shear force

$$F_{v,Rd} = n_o * 2 * 0{,}6 * \frac{f_{u,bo} * \frac{p}{4} * d_{lo}^2}{10^3 * g_{M2}} = 361{,}91\ kN$$

Limit bearing force:

$$a = MIN\left(\frac{a_1}{3 * d_{l1o}}; \frac{a}{3 * d_{l1o}} - 0{,}25; \frac{f_{u,bo}}{f_u}; 1\right) = 0{,}682$$

$$F_{b,Rd} = 2 * n_o * 2{,}5 * a * d_{lo} * t * \frac{f_u}{g_{M2} * 10^3} = 577{,}46\ kN$$

$$MAX\left(\frac{N_{Ed}}{F_{v,Rd}}; \frac{N_{Ed}}{F_{b,Rd}}\right) = \underline{1{,}32 < 1}$$

Analysis of bolts in web:

$$n_{s1} = \frac{n_s - 1}{2} + 0{,}49 \quad = \quad 1$$

$$n_{s2} = n_{s1} * 2 \quad = \quad 2$$

$$m_{s1} = \frac{m_s - 1}{2} + 0{,}49 \quad = \quad 1$$

$$m_{s2} = m_{s1} * 2 \quad = \quad 2$$

$$I_p = \frac{n_s * m_{s2} * \left(\frac{m_s - 1}{2} * e\right)^2 + n_{s2} * m_s * \left(\frac{n_s - 1}{2} * e_3\right)^2}{100} \quad = \quad 420{,}00\ \text{cm}^2$$

$$M_{Ew} = M_w + \frac{V_{Ed} * e_3}{10^3} \quad = \quad 39{,}11\ \text{kNm}$$

$$R_{Ed} = \sqrt{\left(\frac{10 * M_{Ew} * e_3}{I_p}\right)^2 + \left(\frac{V_{Ed}}{m_s * n_s} + 10 * M_{Ew} * \frac{m_s - 1}{2} * \frac{e}{I_p}\right)^2} \quad = \quad 96{,}27\ \text{kN}$$

$$F_{v,Rd} = m_s * 0{,}6 * f_{u,bs} * \frac{p}{4} * \frac{d_{ls}^2}{\gamma_{M2} * 10^3} \quad = \quad 77{,}21\ \text{kN}$$

$$\alpha = \text{MIN}\left(\frac{e_1}{3 * d_{l1s}}; \frac{e}{3 * d_{l1s}} - 0{,}25; \frac{f_{u,bo}}{f_u}; 1\right) \quad = \quad 0{,}833$$

$$F_{b,Rd} = 2{,}5 * \alpha * d_{ls} * s * \frac{f_u}{\gamma_{M2} * 10^3} \quad = \quad 59{,}50\ \text{kN}$$

$$\text{MAX}\left(\frac{R_{Ed}}{F_{v,Rd}}; \frac{R_{Ed}}{F_{b,Rd}}\right) \quad = \quad \underline{1{,}62 < 1}$$

Analysis of cover plates:

$$A = \frac{d_1 * b_1}{100} \quad = \quad 32{,}00\ \text{cm}^2$$

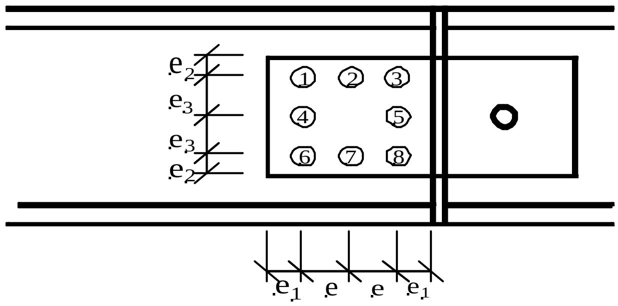
$$A_{net} = A - \frac{2 * d_{l1o} * d_1}{100} \quad = \quad 23{,}20\ \text{cm}^2$$

Limit tension force:

$$N_{t,Rd} = \text{MIN}\left(A * \frac{f_y}{\gamma_{M0} * 10}; 0{,}9 * A_{net} * \frac{f_u}{\gamma_{M2} * 10}\right) \quad = \quad 601{,}34\ \text{kN}$$

$$\frac{N_{Ed}}{N_{t,Rd}} \quad = \quad \underline{0{,}80 < 1}$$

Connection subject to moment load:



Dimensions of connection:

Bolt spacing e =	50,00 mm
Edge distance e1 =	35,00 mm
Bolt spacing e3 =	70,00 mm
Plate thickness t =	6,00 mm
Number of bolts n =	8

Bolts:

Bolt1 =	SEL("steel/bolt"; BS;)	=	M 30
SC1 =	SEL("steel/bolt"; SC;)	=	5.6
$f_{u,b1}$ =	TAB("steel/bolt"; fubk; SC=SC1)	=	500,00 N/mm²
Hole diameter d_{l1} =	TAB("steel/bolt"; d; BS=Bolt1)	=	30,00 mm
Bolt2 =	SEL("steel/bolt"; BS;)	=	M 12
SC2 =	SEL("steel/bolt"; SC;)	=	5.6
$f_{u,b2}$ =	TAB("steel/bolt"; fubk; SC=SC2)	=	500,00 N/mm²
Hole diameter d_{s2} =	TAB("steel/bolt"; d; BS=Bolt2)	=	12,00 mm
Shaft diameter d_{l2}=	d_{s2}+1	=	13,00 mm
steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_{u2} =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
g_{Mb} =	1,25		
g_{Mo} =	1,25		

Loads:

V_{Ed} =	58,70 kN
M_{Ed} =	910,00 kNcm

Analysis of the right bolt:

Check limit shear force

$$F_{v,Rd} = 0{,}6 * \frac{\pi}{4} * \left(\frac{d_{l1}}{10}\right)^2 * \frac{f_{u,b1}}{10 * \gamma_{Mo}} = 169{,}65 \text{ kN}$$

$$\frac{V_{Ed}}{F_{v,Rd}} = \underline{\mathbf{0{,}35 < 1}}$$

Limit bearing strength

$$F_{b,Rd} = 1{,}5 * d_{l1} * t * \frac{f_{u2}}{10^3 * \gamma_{Mb}} = 77{,}76 \text{ kN}$$

$$\frac{V_{Ed}}{F_{b,Rd}} = \underline{\mathbf{0{,}75 < 1}}$$

Check bolts on left:

$$I_p = \frac{6 * \left(e^2 + e_3{}^2\right)}{100} = 444{,}00 \text{ cm}^2$$

Maximum horizontal force in bolt due to moment:

$$F_h = M_{Ed} * \frac{e_3}{I_p * 10} = 14{,}35 \text{ kN}$$

Maximum vertical force:

$$F_v = M_{Ed} * \frac{e}{I_p * 10} + \frac{V_{Ed}}{n} = 17{,}59 \text{ kN}$$

Resulting force

$$R_r = \sqrt{F_h{}^2 + F_v{}^2} = 22{,}70 \text{ kN}$$

$$F_{v,Rd} = 0{,}6 * f_{u,b2} * d_{l2}{}^2 * \frac{\pi}{4 * \gamma_{Mb} * 10^3} = 31{,}86 \text{ kN}$$

$$\frac{R_r}{F_{v,Rd}} = \underline{\mathbf{0{,}71 < 1}}$$

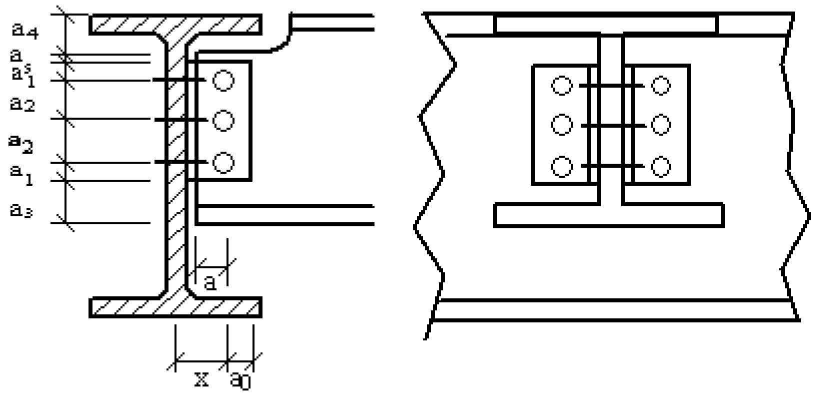
Analysis of bearing strength

$$\alpha = \mathrm{MIN}\left(\frac{e_1}{3 * d_{l2}};\ 2 * \frac{e}{3 * d_{l2}} - 0{,}25;\ \frac{f_{u,b2}}{f_{u2}};\ 1\right) = 0{,}90$$

$$F_{b,Rd} = 2{,}5 * \alpha * d_{l2} * t * \frac{f_{u2}}{\gamma_{Mb} * 10^3} = 50{,}54 \text{ kN}$$

$$\frac{F_h}{F_{b,Rd}} = \underline{\mathbf{0{,}28 < 1}}$$

Bolted shear joint:



Dimensions of connection:

Spacing of bolts a =	45,00 mm
Spacing of bolts a_0 =	35,00 mm
Spacing of bolts a_1 =	30,00 mm
Spacing of bolts a_2 =	50,00 mm
Spacing of bolts a_3 =	90,00 mm
Depth of cope a_4 =	30,00 mm
Plate projection a_s =	20,00 mm
Lever-arm x =	45,00 mm
Number of bolts n =	3

Bolts:

M16 4.6

Bolt =	SEL("steel/bolt"; BS;)	=	M 16
SC =	SEL("steel/bolt"; SC;)	=	4.6
$f_{u,b}$ =	TAB("steel/bolt"; fubk; SC=SC)	=	400,00 N/mm²
Hole diameter d_l=	TAB("steel/bolt"; d; BS=Bolt)	=	16,00 mm
Shaft diameter d_0=	d_l+2	=	18,00 mm

Angle section P =	SEL("steel/WG"; Name;)	=	L 80x8
Angle thickness t =	TAB("steel/WG"; s; Name=P)	=	8,00 mm

Factor for group bolts as in 6.5.2..2(3):

k = 0,5 for 1 group bolts; =2,5 for 2 rows

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²

Profil Typ =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil =	SEL("steel/"Typ; Name;)	=	IPE 270
Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	270,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	6,60 mm

g_{Mb} =	1,25
g_{M2} =	1,25
g_{M0} =	1,10

Loads:

V_{Ed} = 60,00 kN

Check bolt spacing as in 6.5.1:

$1{,}2 * d_0 / a_1$	=	0,72 < 1
a_1 / MAX(12 * t; 150)	=	0,20 < 1
$1{,}5 * d_0 / a_0$	=	0,77 < 1
a_0 / MAX(12 * t; 150)	=	0,23 < 1
$2{,}2 * d_0 / a_2$	=	0,79 < 1
a_2 / MIN(12 * s; 200)	=	0,63 < 1

Check bolts strength:

Bolts in web of main beam $F_{v,Ed}$ = $\dfrac{V_{Ed}}{2 * n}$ = 10,00 kN

Bolts in web of connecting beam M_{Ed} = $\dfrac{x}{10} * V_{Ed}$ = 270,00 kNcm

Horizontal force due to M_{Ed} $F_{h,Ed}$ = $\dfrac{M_{Ed}}{(n - 1) * \dfrac{a_2}{10}}$ = 27,00 kN

Distributed shearing force on bolts $F_{v,Ed}$ = $\dfrac{V_{Ed}}{n}$ = 20,00 kN

Maximum bolt strength F_{Ed} = $\sqrt{F_{h,Ed}^{\ 2} + F_{v,Ed}^{\ 2}}$ = 33,60 kN

Limit strength and analysis of bolts:

Bolts in web of main beam:

Plain in the shear plane $F_{v,Rd}$ = $0{,}6 * f_{u,b} * \pi * \dfrac{d_l^{\ 2}}{4 * \gamma_{Mb} * 10^3}$ = 38,60 kN

Limit bearing strength

α = $\mathrm{MIN}\left(\dfrac{a_1}{3 * d_0}; \dfrac{a_2}{3 * d_0} - 0{,}25; \dfrac{f_{u,b}}{f_u}; 1\right)$ = 0,556

$F_{b,Rd}$ = $2{,}5 * \alpha * d_l * t * \dfrac{f_u}{\gamma_{M2} * 10^3}$ = 51,24 kN

Analysis:

$\dfrac{F_{Ed}}{F_{b,Rd}}$ = **0,66 < 1**

Bolts in web of connecting beam:

$F_{v,Rd}$ = $2 * 0{,}6 * f_{u,b} * \pi * \dfrac{d_l^{\ 2}}{4 * \gamma_{Mb} * 10^3}$ = 77,21 kN

Limit bearing strength

α = $MIN(\frac{a_1}{3 * d_0}; \frac{a_2}{3 * d_0} - 0,25; \frac{f_{u,b}}{f_u}; 1)$ = 0,556

$F_{b,Rd}$ = $2,5 * \alpha * d_l * s * \frac{f_u}{\gamma_{M2} * 10^3}$ = 42,27 kN

Analysis:

$\frac{F_{Ed}}{F_{b,Rd}}$ = **0,79 < 1**

Check angle section strength

Design loads:

V_{Edw} = $\frac{V_{Ed}}{2}$ = 30,00 kN

M_{Edw} = $\frac{M_{Ed}}{2}$ = 135,00 kNm

A = $(2 * a_2 + 2 * a_1) * \frac{t}{200}$ = 6,40 cm²

A_{net} = $(2 * a_2 + 2 * a_1 - n * d_0) * \frac{t}{200}$ = 4,24 cm²

$\frac{\frac{f_y}{f_u} * \frac{\gamma_{M2}}{\gamma_{M0}}}{0,9 * \frac{A_{net}}{A}}$ = **1,24 > 1**

⇒ Subtract bottom bolt holes.

A = A * 2 = 12,80 cm²

A_{net} = A_{net} * 2 = 8,48 cm²

$\frac{f_y}{f_u} * \frac{A}{A_{net}}$ = **0,99 < 1**

⇒ Bolt holes should not be subtracted

W_{pl} = $\frac{t * \frac{(2 * a_2 + 2 * a_1)^2}{4} - d_0 * t * a_2}{1000}$ = 44,00 cm³

$M_{pl,Rd}$ = $W_{pl} * \frac{f_y}{\gamma_{M0} * 10}$ = 940,00 kNcm

$\frac{M_{Ed}}{M_{pl,Rd}}$ = **0,29 < 1**

$V_{pl,Rd} = \quad A * \dfrac{f_y}{\gamma_{M0} * \sqrt{3} * 10} \quad = \quad 157{,}88$ kN

$\dfrac{V_{Ed}}{V_{pl,Rd}} \quad = \quad \underline{\mathbf{0{,}38 < 1}}$

Also:

$\dfrac{V_{Ed}}{\left(\dfrac{V_{pl,Rd}}{2}\right)} \quad = \quad \underline{\mathbf{0{,}76 < 1}}$

$\Rightarrow$ No interaction.

Check shear failure:

$L_v = \quad (n - 1) * a_2 \quad = \quad 100{,}00$ mm

$L_1 = \quad a_2 \quad = \quad 50{,}00$ mm

$\dfrac{L_1}{5 * d_0} \quad = \quad \underline{\mathbf{0{,}56 < 1}}$

$L_2 = \quad (a - k * d_0) * \dfrac{f_u}{f_y} \quad = \quad 55{,}15$ mm

$L_3 = \quad L_v + a_2 + a_3 \quad = \quad 240{,}00$ mm

$L_{v,eff} = \quad L_v + L_1 + L_2 \quad = \quad 205{,}15$ mm

$\dfrac{L_{v,eff}}{L_3} \quad = \quad \underline{\mathbf{0{,}85 < 1}}$

$\dfrac{L_3}{(L_v + a_2 + a_3 - n * d_0) * \dfrac{f_u}{f_y}} \quad = \quad \underline{\mathbf{0{,}84 < 1}}$

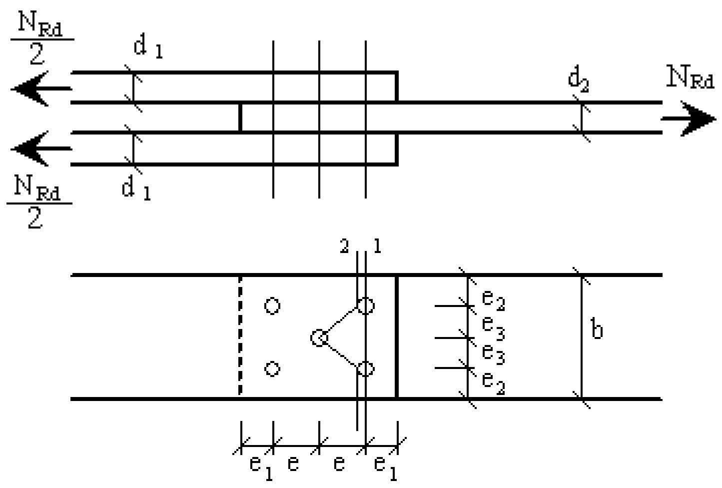
Effective shear area:

$A_{v,eff} = \quad L_{v,eff} * t / 100 \quad = \quad 16{,}41$ cm²

$V_{eff,Rd} = \quad A_{v,eff} * \dfrac{f_y}{\gamma_{M0} * \sqrt{3} * 10} \quad = \quad 202{,}41$ kN

$\dfrac{V_{Ed}}{V_{eff,Rd}} \quad = \quad \underline{\mathbf{0{,}30 < 1}}$

Double shear joint with bolts:



Dimensions of connection:

Plate thickness d_1 =	6,00 mm
Number of plates n_1 =	2
Plate thickness d_2 =	7,10 mm
Number of plates n_2 =	1
Plate width b =	210,00 mm
Bolt spacing e =	40,00 mm
Edge distance e1 =	60,00 mm
Edge distance e2 =	40,00 mm
Bolt spacing e3 =	65,00 mm

Bolts:

Bolt =	SEL("steel/bolt"; BS;)	=	M 20
SC =	SEL("steel/bolt"; SC;)	=	10.9
$f_{u,b}$ =	TAB("steel/bolt"; fubk; SC=SC)	=	1000,00 N/mm²
Hole diameter d_l=	TAB("steel/bolt"; d; BS=Bolt)	=	20,00 mm
Cross-section areas A_s =	TAB("steel/bolt"; Asp; BS=Bolt)	=	2,45 cm²
Number of bolts n=			5

Plate :

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
g_{M0} =	1,10		
g_{M2} =	1,25		

Loads:

N_{Ed} =	100,00 kN

Joint with tension and shear:

Limit tension force of plate links

$A = b * d_1 * \frac{n_1}{100}$ = 25,20 cm²

$d = d_l + 2$ = 22,00 mm

Cross-section area 1:

$A_{net,1} = A - 2 * d * d_1 * \frac{n_1}{100}$ = 19,92 cm²

Cross-section area 2:

$A_{net,2} = A\text{-}3 * d * d_1 * \frac{n_1}{100} + 2 * e^2 / (4*e_3) * d_1 * \frac{n_1}{100}$ = 18,76 cm²

$A_{net} = MIN(A_{net,1}; A_{net,2})$ = 18,76 cm²

$N_{t,Rd1} = MIN(A * \frac{f_y}{g_{M0}}; 0,9 * A_{net} * \frac{f_u}{g_{M2}})/10$ = 486,26 kN

Limit tension force of plate rechts

$A = b * d_2 * \frac{n_2}{100}$ = 14,91 cm²

$d = d_l + 2$ = 22,00 mm

$A_{net,1} = A - 2 * d * d_2 * \frac{n_2}{100}$ = 11,79 cm²

$A_{net,2} = A - 3 * d * d_2 * \frac{n_2}{100} + 2 * e^2 / (4*e_3) * d_2 * \frac{n_2}{100}$ = 11,10 cm²

$A_{net} = MIN(A_{net,1}; A_{net,2})$ = 11,10 cm²

$N_{t,Rd2} = MIN(A * \frac{f_y}{g_{M0}}; 0,9 * A_{net} * \frac{f_u}{g_{M2}})/10$ = 287,71 kN

$N_{t,Rd} = MIN(N_{t,Rd2}; N_{t,Rd1})$ = **287,71 kN**

Limit tension force of boltsn:

$A_s = \frac{p * \left(\frac{d_l}{10}\right)^2}{4}$ = 3,14 cm²

Thread in the shearing plane:

$F_{v,Rd} = 0,6 * n * (n_1 + n_2 - 1) * \frac{f_{u,b} * A_s}{10 * g_{M2}}$ = 1507,20 kN

Limit bearing strength

$a = MIN(\frac{e_1}{3 * d}; 2 * \frac{e}{3 * d} - 0,25; \frac{f_{u,b}}{f_u}; 1)$ = 0,91

$F_{b,Rd1} = n_1 * n * 2,5 * a * d_l * d_1 * \frac{f_u}{g_{M2} * 10^3}$ = 786,24 kN

$a = \quad MIN(\frac{e_1}{3 * d}; 2 * \frac{e}{3 * d} - 0{,}25; \frac{f_{u,b}}{f_u}; 1) \quad = \quad 0{,}91$

$F_{b,Rd2} = \quad n_2 * n * 2{,}5 * a * d_l * d_2 * \frac{f_u}{g_{M2} * 10^3} \quad = \quad 465{,}19$ kN

$F_{b,Rd} = \quad MIN(F_{b,Rd1}; F_{b,Rd2}) \quad =$ **465,19 kN**

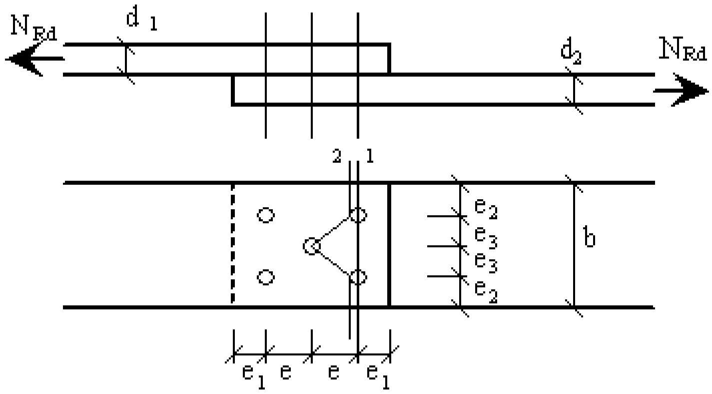
Maximum allowed force:

$N_{l,Rd} = \quad MIN(N_{t,Rd}; F_{v,Rd}; F_{b,Rd}) \quad =$ **287,71 kN**

Analysis:

$N_{Ed} / N_{l,Rd} \quad =$ **0,35 < 1**

High strength friction-grip bolt connection:



Dimensions of connection:

Plate thickness 1 d_1 =	10,00 mm
Plate thickness 2 d_2 =	8,00 mm
Bolt spacing e =	65,00 mm
Edge distance e1 =	55,00 mm
Edge distance e2 =	50,00 mm
Bolt spacing e3 =	70,00 mm
Plate width b=	240,00 mm

Bolts:

Bolt =	SEL("steel/bolt"; BS;)	=	M 22
SC =	SEL("steel/bolt"; SC;)	=	10.9
$f_{u,b}$ =	TAB("steel/bolt"; fubk; SC=SC)	=	1000,00 N/mm²
Hole diameter d_l=	TAB("steel/bolt"; d; BS=Bolt)	=	22,00 mm
Cross-section areas A_s=	TAB("steel/bolt"; Asp; BS=Bolt)	=	3,03 cm²
Number of bolts n=			5

normal holes with tolerance as K_s =	1,00
Number of slip joints as in 6.5.8.1 (1) h =	1,00
coefficient of friction as in 6.5.8.3 m =	0,40
as in 6.5.8.1 (3) g_{Ms} =	1,25

Plate:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
g_{M0} =			1,10
g_{M2} =			1,25

Loads:

Tension force N_{Ed} =	100,00 kN

Limit tension force of plate:

$A = \mathrm{MIN}(d_1;\, d_2) * \dfrac{b}{100}$ = 19,20 cm²

as in 7.5.2 $d = d_l + 2$ = 24,00 mm

Cross-section area 1:

$A_{net,1} = A - 2 * \mathrm{MIN}(d_1;\, d_2) * \dfrac{d}{100}$ = 15,36 cm²

Cross-section area 2:

$A_{net,2} = A - 3 * \dfrac{d}{100} * \mathrm{MIN}(d_1;d_2) + \dfrac{2}{100} * \dfrac{e^2}{4 * e_3} * \mathrm{MIN}(d_1;d_2)$ = 15,85 cm²

$A_{net} = \mathrm{MIN}(A_{net,1};\, A_{net,2})$ = 15,36 cm²

Friction-grip connection:

Plate

$N_{net,Rd} = A_{net} * \dfrac{f_y}{g_{M0} * 10}$ = 328,15 kN

Limit tension force of bolts

Limit shank tension $F_{p,cd} = 0{,}7 * \dfrac{f_{u,b}}{10} * A_s$ = 212,10 kN

Limit slip resistance force $F_{s,Rd} = n * K_s * h * m * \dfrac{F_{p,cd}}{g_{Ms}}$ = 339,36 kN

Limit bearing force:

$a = \mathrm{MIN}(\dfrac{e_1}{3 * d};\, 2 * \dfrac{e}{3 * d} - 0{,}25;\, \dfrac{f_{u,b}}{f_u};\, 1)$ = 0,76

$F_{b,Rd} = n * 2{,}5 * a * d_l * \mathrm{MIN}(d_1;\, d_2) * \dfrac{f_u}{g_{M2} * 10^3}$ = 481,54 kN

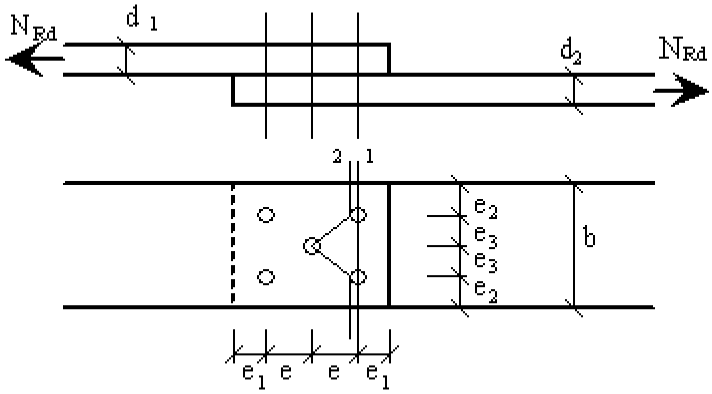
Maximum allowed force:

$N_{g,Rd} = \mathrm{MIN}(N_{net,Rd};\, F_{b,Rd};\, F_{s,Rd})$ = **328,15 kN**

Analysis:

$\dfrac{N_{Ed}}{N_{g,Rd}}$ = **0,30 < 1**

Joint with tension and shear:



Dimensions of connection:

Plate thickness d_1 =	10,00 mm
Plate thickness d_2 =	8,00 mm
Bolt spacing e =	65,00 mm
Edge distance e1 =	55,00 mm
Edge distance e2 =	50,00 mm
Bolt spacing e3 =	70,00 mm
Plate width b =	240,00 mm

Bolts:

Bolt =	SEL("steel/bolt"; BS;)	=	M 22
SC =	SEL("steel/bolt"; SC;)	=	4.6
$f_{u,b}$ =	TAB("steel/bolt"; fubk; SC=SC)	=	400,00 N/mm²
Hole diameter d_l =	TAB("steel/bolt"; d; BS=Bolt)	=	22,00 mm
Cross-section areas A_s =	TAB("steel/bolt"; Asp; BS=Bolt)	=	3,03 cm²
Number of bolts n =			5

Plate:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
g_{M0} =	1,10		
g_{M2} =	1,25		

Loads:

N_{Ed} = 100,00 kN

Joint with tension and shear:

Limit tension force of plate:

$A = \text{MIN}(d_1;\ d_2) * \frac{b}{100}$ = 19,20 cm²

$d = d_l + 2$ = 24,00 mm

Cross-section area 1:

$A_{net,1} = A - 2 * \text{MIN}(d_1;\ d_2) * \frac{d}{100}$ = 15,36 cm²

Cross-section area 2:

$A_{net,2} = A - 3 * \frac{d}{100} * MIN(d_1;d_2) + \frac{2}{100} * \frac{e^2}{4 * e_3} * MIN(d_1;d_2)$ = 15,85 cm²

$A_{net} = MIN(A_{net,1};\ A_{net,2})$ = 15,36 cm²

$N_{t,Rd} = MIN(\ A * \frac{f_y}{g_{M0}};\ 0,9 * A_{net} * \frac{f_u}{g_{M2}})/10$ = 398,13 kN

Limit tension force of boltsn:

$A_s = \frac{p * \left(\frac{d}{10}\right)^2}{4}$ = 4,52 cm²

Thread in the shearing plane:

$F_{v,Rd} = 0,6 * n * \frac{f_{u,b} * A_s}{10 * g_{M2}}$ = 433,92 kN

Limit bearing strength

$a = MIN(\frac{e_1}{3 * d};\ 2 * \frac{e}{3 * d} - 0,25;\ \frac{f_{u,b}}{f_u};\ 1)$ = 0,76

$F_{b,Rd} = n * 2,5 * a * d * MIN(d_1;\ d_2) * \frac{f_u}{g_{M2} * 10^3}$ = 525,31 kN

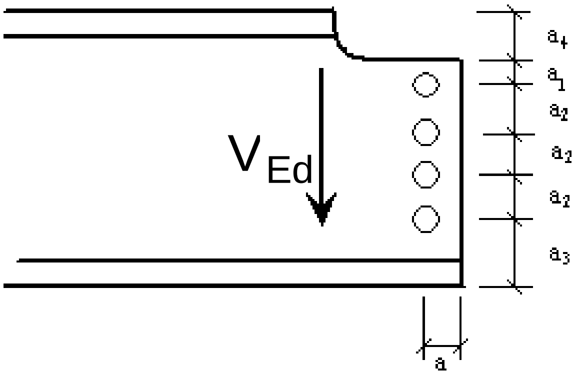
Maximum allowed force:

$N_{l,Rd} = MIN(N_{t,Rd};\ F_{v,Rd};\ F_{b,Rd})$ = **398,13 kN**

Analysis:

$N_{Ed} / N_{l,Rd}$ = **0,25 < 1**

Check shear failure:



Dimensions of connection:

Spacing of bolts a =	50,00 mm
Spacing of bolts a_1 =	43,00 mm
Spacing of bolts a_2 =	70,00 mm
Spacing of bolts a_3 =	75,00 mm
Number of bolts n =	4

Bolts:

Bolt =	SEL("steel/bolt"; BS;)	=	M 20
SC =	SEL("steel/bolt"; SC;)	=	5.6
$f_{u,b}$ =	TAB("steel/bolt"; fubk; SC=SC)	=	500,00 N/mm²
Hole diameter d_l=	TAB("steel/bolt"; d; BS=Bolt)	=	20,00 mm
Shaft diameter d_{l2}=	d_l+2	=	22,00 mm
k =	0,5 for 1 group bolts; =2,5 for 2 rows		

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²

Profil Typ =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil =	SEL("steel/"Typ; Name;)	=	IPE 360
Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	360,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	8,00 mm
γ_{Mb} =	1,25		
γ_{M0} =	1,10		

Loads:

V_{Ed} = 58,70 kN

Limit shear force of section:

$$A_v = 1{,}04 * \frac{h * s}{100} = 29{,}95 \text{ cm}^2$$

$$V_{pl,Rdp} = A_v * \frac{f_y}{\sqrt{3} * \gamma_{M0} * 10} = 369{,}41 \text{ kN}$$

$A_{v,net} = \quad A_v - \frac{n * s * d_{l2}}{100} \quad = \quad 22,91$

$\frac{f_y}{f_u} * \frac{A_v}{A_{v,net}} \quad = \quad \underline{\mathbf{0,85 < 1}}$

Þ Holes can be neglected.

Joint with shear failure as in Bild 6.5.5:

$L_v = \quad (n - 1) * a_2 \quad = \quad 210,00 \text{ mm}$

$L_1 = \quad a_1 \quad = \quad 43,00 \text{ mm}$

$\frac{L_1}{5 * d_{l2}} \quad = \quad 0,39 < 1$

$L_2 = \quad (a - k * d_{l2}) * \frac{f_u}{f_y} \quad = \quad 59,74 \text{ mm}$

$L_3 = \quad L_v + a_1 + a_3 \quad = \quad 328,00 \text{ mm}$

$L_{v,eff} = \quad L_v + L_1 + L_2 \quad = \quad 312,74 \text{ mm}$

$\frac{L_{v,eff}}{L_3} \quad = \quad \underline{\mathbf{0,95 < 1}}$

$\frac{L_3}{L_v + a_1 + a_3 - n + d_{l2}} \quad = \quad \underline{\mathbf{0,95 < 1}}$

Effective shear area:

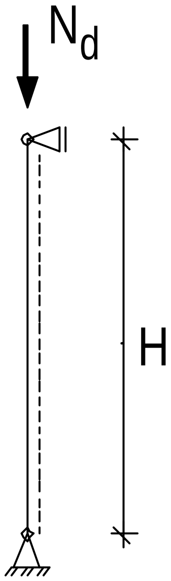
$A_{v,eff} = \quad L_{v,eff} * \frac{s}{100} \quad = \quad 25,02 \text{ cm}^2$

$V_{pl,Rd} = \quad A_{v,eff} * \frac{f_y}{g_{M0} * Ö3 * 10} \quad = \quad 308,60 \text{ kN}$

Maximum design force for shear:

$V_{Rd} = \quad MIN(V_{pl,Rdp}; V_{pl,Rd}) \quad = \quad \underline{\mathbf{308,60 \text{ kN}}}$

Column subject to compression force:



Column heigth H =			7,50 m
l_y =	H/2	=	3,75 m
l_z =	H/3	=	2,50 m

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 430
f_y =	TAB("steel/EC"; fy; Name=steel)	=	275,00 N/mm²
E =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
ε =	$\sqrt{\frac{235}{f_y}}$	=	0,92

Profil:

Profil Typ =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil =	SEL("steel/"Typ; Name;)	=	IPE 300
Cross-sectional area A =	TAB("steel/"Typ; A; Name=Profil)	=	53,80 cm²
Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	300,00 mm
Depth of web h_1 =	TAB("steel/"Typ; h1; Name=Profil)	=	248,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	7,10 mm
Flange width b =	TAB("steel/"Typ; b; Name=Profil)	=	150,00 mm
Flange thickness t =	TAB("steel/"Typ; t; Name=Profil)	=	10,70 mm
Radius of gyration i_y =	TAB("steel/"Typ; iy; Name=Profil)	=	12,50 cm
Radius of gyration i_z=	TAB("steel/"Typ; iz; Name=Profil)	=	3,35 cm

Section classification As in Table 5.3.1:

Web:

$$\frac{h_1}{s * 33 * \varepsilon} = 1,15 < 1$$

$$\frac{h_1}{s * 38 * \varepsilon} = 1,00 < 1$$

Section class 2.

Flange:

$$\frac{b}{2 * t * 10 * \varepsilon} = 0{,}76 < 1$$

Section class 1.
Section will be classified as class 2.

<u>Analysis:</u>

nach 5.5.1.1 (1) β_A= 1,00 Cross-section class 2
nach 5.1.1 (2) γ_{M1}= 1,10 Cross-section class 2

Check buckling In Y-Y Axis:

h / b = 2,00 > 1,2
t /10 = 1,07 cm < 4 cm
as in Tab 5.5.3 :

$$\lambda_y = l_y * \frac{100}{i_y} = 30{,}00$$

$$\lambda_1 = 93{,}9 * \varepsilon = 86{,}39$$

$$\lambda_{trans,y} = \frac{\lambda_y}{\lambda_1} * \sqrt{\beta_A} = 0{,}347$$

Apply strut curve a
As in Table 5.5.1 α = 0,21

$$\phi = 0{,}5 * (1 + \alpha * (\lambda_{trans,y} - 0{,}2) + \lambda_{trans,y}^2) = 0{,}576$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans,y}^2}} = 0{,}9655$$

$$N_{b,y,Rd} = \chi * \beta_A * A * \frac{f_y}{10 * \gamma_{M1}} = 1298{,}60 \text{ kN}$$

Check buckling In Z-Z Axis:

h / b = 2,00 > 1,2
t / 10 = 1,07 cm < 4cm
as in Tab 5.5.3 :

$$\lambda_z = l_z * 100 / i_z = 74{,}63$$

$$\lambda_{trans,z} = \frac{\lambda_z}{\lambda_1} * \sqrt{\beta_A} = 0{,}864$$

Apply strut curve b
As in Table 5.5.1 α = 0,34

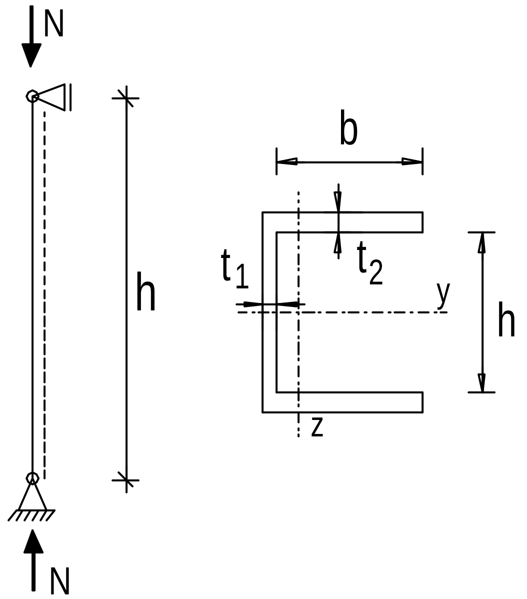
$$\phi = 0{,}5 * (1 + \alpha * (\lambda_{trans,z} - 0{,}2) + \lambda_{trans,z}^2) = 0{,}986$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans,z}^2}} = 0{,}6844$$

$$N_{b,z,Rd} = \chi * \beta_A * A * \frac{f_y}{10 * \gamma_{M1}} = 920{,}52 \text{ kN}$$

$$max\_N_d = MIN(N_{b,y,Rd}; N_{b,z,Rd}) = \underline{920{,}52 \text{ kN}}$$

Column of section class 4:



Load diagram:

Column heigth H =	4,00 kN
Flange width b =	20,00 cm
Depth of web h =	30,00 cm
Web thickness t_1 =	1,20 cm
Flange thickness t_2 =	1,20 cm

Loads:

N_d = 700,00 kN

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
E =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
$\varepsilon =$	$\sqrt{\dfrac{235}{f_y}}$	=	1,00
λ_1 =	93,90 * ε	=	93,90

Partial safety factors:

γ_M = 1,10

Section classification As in Table 5.3.1:

$$\frac{b}{t_2 * 14 * \varepsilon} = \underline{\mathbf{1,19 > 1}}$$

Section class 4

$$\frac{h}{t_1 * 33 * \varepsilon} = \underline{\mathbf{0,76 > 1}}$$

Section class 1

Effective web area:

Flangee:

As in Table 5.3.3:

ψ = 1,00

k_s = 0,43

as in 5.3.5(3)

$$\lambda_{trans,p} = \frac{b}{t_2 * 28{,}4 * \varepsilon * \sqrt{k_s}} = 0{,}89 > 0{,}673$$

$$\rho = \frac{\lambda_{trans,p} - 0{,}22}{\lambda_{trans,p}^{2}} = 0{,}846$$

$$b_{eff} = \rho * b = 16{,}92 \text{ cm}$$

$$A_{eff} = 2 * (b_{eff} * t_2) + (h + 2 * t_2) * t_1 = 79{,}49 \text{ cm}^2$$

Location of shear centre:

$$y_{s,eff} = \frac{b_{eff} * 2 * t_2 * \dfrac{b_{eff} + t_2}{2}}{A_{eff}} = 4{,}628 \text{ cm}$$

$$I_{z,eff} = \frac{b_{eff}^{3}}{12} * 2 * t_2 + b_{eff} * 2 * t_2 * \left(\frac{b_{eff} + t_1}{2} - y_{s,eff}\right)^2 + (h + 2 * t_2) * \left(\frac{t_1^{3}}{12} + t_1 * y_{s,eff}^{2}\right) = 2604 \text{ cm}^4$$

$$W_{z,eff} = \frac{I_{z,eff}}{(b_{eff} + t_1) - y_{s,eff} - \dfrac{t_1}{2}} = 201{,}99 \text{ cm}^3$$

Gross area:

$$A = b * 2 * t_2 + (h + 2 * t_2) * t_1 = 86{,}88 \text{ cm}^2$$

Location of center of gravity:

$$y_s = \frac{b * 2 * t_2 * \dfrac{b + t_1}{2}}{A} = 5{,}856 \text{ cm}$$

$$I_z = \frac{b^3}{12} * 2 * t_2 + b * 2 * t_2 * \left(\frac{b + t_1}{2} - y_s\right)^2 + (h + 2 * t_2) * \left(\frac{t_1^{3}}{12} + t_1 * y_s^{2}\right) = 4018 \text{ cm}^4$$

$$e_{Nz} = y_s - y_{s,eff} = 1{,}228 \text{ cm}$$

Check buckling:

$$N_{cr} = \pi^2 \cdot \frac{E}{10} \cdot \frac{I_z}{(H \cdot 100)^2} = 5204{,}85 \text{ kN}$$

$$\beta_A = \frac{A_{eff}}{A} = 0{,}915$$

$$\lambda_{trans,z} = \sqrt{\beta_A \cdot A \cdot \frac{f_y}{10 \cdot N_{cr}}} = 0{,}599$$

Apply strut curve c As in Table 5.5.3

$$\alpha = 0{,}49$$

$$\phi = 0{,}5 \cdot (1 + \alpha \cdot (\lambda_{trans,z} - 0{,}2) + \lambda_{trans,z}^2) = 0{,}777$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans,z}^2}} = 0{,}786$$

$$M_{z,d} = N_d \cdot \frac{e_{Nz}}{100} = 8{,}60 \text{ kNm}$$

$$\psi = 1{,}00$$

$$\beta_{My} = 1{,}8 - 0{,}7 \cdot \psi = 1{,}10$$

$$\mu_z = \lambda_{trans,z} \cdot (2 \cdot \beta_{My} - 4) = -1{,}078 < 0{,}9$$

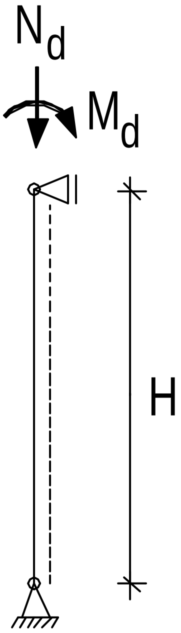
$$k_{z1} = 1 - \frac{\mu_z \cdot N_d}{\chi \cdot A_{eff} \cdot \frac{f_y}{10}} = 1{,}514 \sim 1{,}5$$

$$k_{z2} = 1{,}500$$

$$k_z = \mathrm{MIN}(k_{z1};\ k_{z2}) = 1{,}500$$

$$\frac{N_d}{\chi \cdot A_{eff} \cdot \frac{f_y}{\gamma_M \cdot 10}} + 100 \cdot k_z \cdot \frac{M_{z,d}}{W_{z,eff} \cdot \frac{f_y}{\gamma_M \cdot 10}} = \underline{\mathbf{0{,}823 < 1}}$$

Column with compression and moment:



Load diagram:

Column heigth H =	5,00 m

Loads:

N_d =	200,00 kN
$M_{y,d}$ =	10,00 kN

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
E =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
G =	TAB("steel/EC"; G; Name=steel)	=	81000,00 N/mm²
e =	Ö(235 / f_y)	=	1,00

Partial safety factors:

g_M =	1,10
g_g =	1,35
platischer Formbeiwert a_{ply} =	1,14
platischer Formbeiwert a_{plz} =	1,25

Profil:

Profil Typ =	SEL("steel/Profils"; Name;)	=	HEA
Selected Profil =	SEL("steel/"Typ; Name;)	=	HEA 160
Cross-sectional area A =	TAB("steel/"Typ; A; Name=Profil)	=	38,80 cm²
Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	152,00 mm
Depth of web h_1 =	TAB("steel/"Typ; h1; Name=Profil)	=	104,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	6,00 mm
Flange width b =	TAB("steel/"Typ; b; Name=Profil)	=	160,00 mm
Flange thickness t =	TAB("steel/"Typ; t; Name=Profil)	=	9,00 mm
Radius of gyration i_y =	TAB("steel/"Typ; iy; Name=Profil)	=	6,57 cm
Radius of gyration i_z=	TAB("steel/"Typ; iz; Name=Profil)	=	3,98 cm
W_{ely}=	TAB("steel/"Typ; Wy; Name=Profil)	=	220,00 cm³
W_{ply} =	$a_{ply} * W_{ely}$	=	250,80 cm³
W_{elz}=	TAB("steel/"Typ; Wz; Name=Profil)	=	76,90 cm³
W_{plz} =	$a_{plz} * W_{elz}$	=	96,13 cm³

Section classification As in Table5.3.1:

Web will be assumed as pressed in

$$\frac{h_1}{s * 33 * e} = 0,53 < 1$$

Section class 1.

Flange:

$$\frac{b}{2 * t * 10 * e} = 0,89 < 1$$

Section class 1.

Check bending and buckling:

as in 5.5.1.1 (1) b_A = 1,00 Cross-section class 1

as in 5.1.1 (2) g_{M1} = 1,10 Cross-section class 1

$$l_y = H * \frac{100}{i_y} = 76,10$$

$$l_1 = 93,9 * e = 93,90$$

$$l_{trans,y} = \frac{l_y}{l_1} * \sqrt{b_A} = 0,810$$

$$l_z = H * \frac{100}{i_z} = 125,63$$

$$l_{trans,z} = \frac{l_z}{p * \sqrt{\frac{E}{f_y}}} = 1,34$$

As in Table 5.5.3

h / b = 0,95 < 1,2

t / 10 = 0,90 cm < 10cm

As in Table:5.5.1

Strut curve b for Y-Y axis

a = 0,34

$$j = 0,5 * (1 + a * (l_{trans,y} - 0,2) + l_{trans,y}^2) = 0,932$$

$c_y = \dfrac{1}{j + \sqrt{j^2 - l_{trans,y}^2}}$ = 0,7179

Apply strut curve c for z-Achse

a = 0,49

j = 0,5 * (1 + a * ($l_{trans,z}$ - 0,2) + $l_{trans,z}^2$) = 1,677

$c_z = \dfrac{1}{j + \sqrt{j^2 - l_{trans,z}^2}}$ = 0,3724

y = 0,00

b_{My} = 1,8 - 0,7 * y = 1,80

$m_y = l_{trans,y} * (2 * b_{My} - 4) + \dfrac{W_{ply} - W_{ely}}{W_{ely}}$ = -0,184 < 0,9

$k_y = 1 - \dfrac{m_y * N_d}{c_y * A * f_y}$ = 1,006 < 1,5

c = MIN(c_y; c_z) = 0,372

$$\frac{N_d}{c * A * \frac{f_y}{g_M * 10}} + k_y * 100 * \frac{M_{y,d}}{W_{ply} * \frac{f_y}{g_M * 10}}$$ = **0,836 < 1**

Check torsional-flexural buckling:

b_w = 1,00 Cross-section class 1

As in Table: F.1.1

y = 0,00

k = 1,00

C_1 = 1,879

$$l_{LT} = 90 * \frac{H}{\sqrt{i_z * \left(\sqrt{C_1} * \sqrt[4]{1 + \frac{1}{20} * \left(\frac{100 * \frac{H}{i_z}}{\frac{h}{t}}\right)^2}\right)}}$$ = 59,208

$l_{trans,LT} = \dfrac{l_{LT}}{l_1} * \sqrt{b_w}$ = 0,631

For rolled sections strut curve a

a = 0,21

j = 0,5 * (1 + a * ($l_{trans,LT}$ - 0,2) + $l_{trans,LT}^2$) = 0,74

$c_{LT} = \dfrac{1}{j + \sqrt{j^2 - l_{trans,LT}^2}}$ = 0,89

$b_{M,LT}$ = 1,8 - 0,7 * y = 1,80

$\lambda_{quer,z} = \dfrac{\lambda_z}{\lambda_1} * \sqrt{\beta_A}$ = 1,338

$\mu_{LT} =$ 0,15 * $\lambda_{quer,z}$ * $\beta_{M,LT}$ - 0,15 = 0,211 < 0,9

h / b = 0,95 < 1,2

t / 10 = 0,90 cm < 10cm

Z-Z axis Strut curve c

$\alpha =$ 0,49

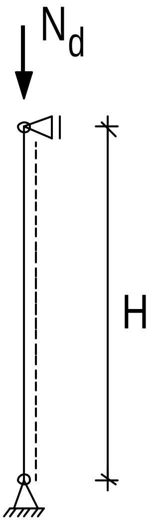
$\varphi =$ 0,5 * (1 + α * ($\lambda_{trans,z}$ - 0,2) + $\lambda_{trans,z}^2$) = 1,677

$\chi_z = \dfrac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{trans,z}^2}}$ = 0,3724

$k_{LT} = 1 - \dfrac{\mu_{LT} * N_d}{\chi_z * A * f_y}$ = 0,988 < 1,5

$\dfrac{N_d}{\chi_z * A * \dfrac{f_y}{\gamma_M * 10}} + k_{LT} * 100 * \dfrac{M_{y,d}}{\chi_{LT} * W_{ply} * \dfrac{f_y}{\gamma_M * 10}}$ = **0,855 < 1**

Column subject to compression force:



Column base is pinned.
For buckling, column head is pinned about the Y-Y axis, and fixed about the Z-Z axis .

Load diagram:

Column heigth H = 8,00 m

Loads:

N_d = 2000,00 kN

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
E =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
ε =	$\sqrt{\frac{235}{f_y}}$	=	1,00

Effective length factor:

k_y = 1,00 For simply supported ends

k_z = 0,70 fixed ends

Profil:

Profil Typ =	SEL("steel/Profils"; Name;)	=	HEB
Selected Profil =	SEL("steel/"Typ; Name;)	=	HEB 300
Cross-sectional area A =	TAB("steel/"Typ; A; Name=Profil)	=	149,00 cm²
Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	300,00 mm
Depth of web h_1 =	TAB("steel/"Typ; h1; Name=Profil)	=	208,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	11,00 mm
Flange width b =	TAB("steel/"Typ; b; Name=Profil)	=	300,00 mm
Flange thickness t =	TAB("steel/"Typ; t; Name=Profil)	=	19,00 mm
Moment of inertia I =	TAB("steel/"Typ; Iy; Name=Profil)	=	25170,00 cm⁴
Radius of gyration i_y =	TAB("steel/"Typ; iy; Name=Profil)	=	13,00 cm
Radius of gyration i_z=	TAB("steel/"Typ; iz; Name=Profil)	=	7,58 cm

Section classification:

Web:

$$\frac{h_1}{s * 33 * e} = 0{,}57 > 1$$

Flange:

$$\frac{b}{2 * t * 10 * e} = 0{,}79 > 1$$

Section class 1.

Analysis:

$b_A =$ 1,00 Cross-section class 1

$g_{M1} =$ 1,10 Cross-section class 1

Check buckling In Y-Y Axis:

h / b = 1,00 < 1,2

t / 10 = 1,90 cm < 10cm

$$l_y = k_y * H * \frac{100}{i_y} = 61{,}54$$

$$l_1 = 93{,}9 * e = 93{,}90$$

$$l_{trans,y} = \frac{l_y}{l_1} * Ö\overline{b_A} = 0{,}655$$

Apply strut curve b

As in Table 5.5.1 $a =$ 0,34

$$j = 0{,}5 * (1 + a * (l_{trans,y} - 0{,}2) + l_{trans,y}{}^2) = 0{,}792$$

$$c = \frac{1}{j + Ö\overline{j^2 - l_{trans,y}{}^2}} = 0{,}8083$$

$$N_{b,Rd} = c * b_A * A * \frac{f_y}{10 * g_{M1}} = 2572{,}966 \text{ kN}$$

$$\frac{N_d}{N_{b,Rd}} = \underline{\mathbf{0{,}78 < 1}}$$

Check buckling In Z-Z Axis:

h / b = 1,00 < 1,2

t / 10 = 1,90 cm < 10cm

$\lambda_z = k_z * H * \frac{100}{i_z}$ = 73,88 m

$\lambda_{trans,z} = \frac{\lambda_z}{\lambda_1} * \sqrt{\beta_A}$ = 0,787

Apply strut curve c

As in Table 5.5.1 α = 0,49

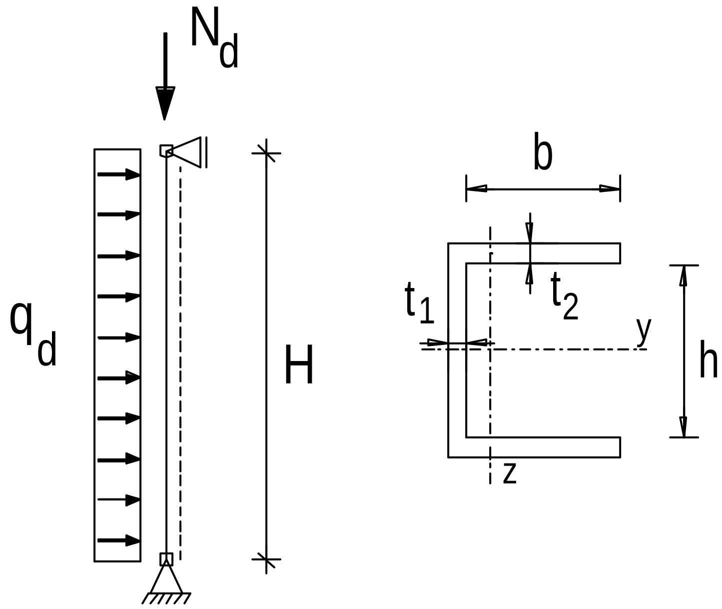
$\phi = 0,5 * (1 + \alpha * (\lambda_{trans,z} - 0,2) + \lambda_{trans,z}^2)$ = 0,953

$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans,z}^2}}$ = 0,6709

$N_{b,Rd} = \chi * \beta_A * A * \frac{f_y}{10 * \gamma_{M1}}$ = 2135,60 kN

$\frac{N_d}{N_{b,Rd}}$ = **0,94 < 1**

Column of channel section with compression and transverse loads:



Load diagram:

Column height H =	4,00 m
Beam width b =	9,50 cm
Depth of web h =	29,40 cm
Web thickness t_1 =	1,00 cm
Flange width t_2 =	0,60 cm

Loads:

q_d =	15,00 kN/m
N_d =	90,00 kN

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
E =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
$\varepsilon =$	$\sqrt{\frac{235}{f_y}}$	=	1,00
l_1 =	93,90 * ε	=	93,90

Partial safety factors:

g_M = 1,10

<u>Section classification:</u>

As in Table 5.3.1

$$\frac{b}{t_2 * 14 * e} = 1{,}13 > 1$$

Section class 4

As in Table 5.3.2

$$\frac{h}{t_1 * 33 * e} = 0{,}89 > 1$$

Section class 1

$A = b * 2 * t_2 + (h + 2 * t_2) * t_1 = 42{,}00\ cm^2$

Location of center of gravity:

$$y_s = \frac{b * 2 * t_2 * \frac{b + t_1}{2}}{A} = 1{,}425\ cm$$

$$I_z = \frac{b^3}{12} * 2 * t_2 + b * 2 * t_2 * \left(\frac{b + t_1}{2} - y_s\right)^2 + (h + 2 * t_2) * \left(\frac{t_1^{\ 3}}{12} + t_1 * y_s^{\ 2}\right) = 317{,}21\ cm^4$$

$$I_y = \frac{h^3}{12} * t_1 + 2 * b * \left(\frac{t_2^{\ 3}}{12} + t_2 * \left(\frac{h + t_2}{2}\right)^2\right) = 4683{,}02\ cm^4$$

$$W_{el,z} = \frac{I_z}{b + \frac{t_1}{2} - y_s} = 36{,}99\ cm^3$$

$$W_{el,y} = \frac{I_y}{\left(\frac{h + t_1}{2}\right)} = 308{,}09\ cm^3$$

<u>Effective web area:</u>

Flangee:

As in Table 5.3.3:

y = 1,00

k_s = 0,43

as in 5.3.5(3)

$$l_{trans,p} = \frac{b}{t_2 * 28{,}4 * e * Ö\, k_s} = 0{,}85 > 0{,}673$$

$$r = \frac{l_{trans,p} - 0{,}22}{l_{trans,p}^{\ 2}} = 0{,}872$$

$b_{eff} = r * b = 8{,}28\ cm$

$A_{eff} = 2 * (b_{eff} * t_2) + (h + 2 * t_2) * t_1 = 40{,}54\ cm^2$

Location of shear centre:

$$y_{s,eff} = \frac{b_{eff} * 2 * t_2 * \dfrac{b_{eff} + t_2}{2}}{A_{eff}} = 1{,}088 \text{ cm}$$

$$e_{Nz} = y_s - y_{s,eff} = 0{,}337 \text{ cm}$$

$$\beta_A = \frac{A_{eff}}{A} = 0{,}965$$

Slenderness and reductions:
In Z-Z Axis:

$$N_{cr,z} = \pi^2 * E * \frac{I_z}{(H * 100)^2} = 4109{,}09 \text{ kN}$$

$$\lambda_{trans,z} = \sqrt{\beta_A * A * \frac{f_y}{N_{cr,z}}} = 1{,}52$$

Apply strut curve c

$$\alpha = 0{,}49$$

$$\phi = 0{,}5 * (1 + \alpha * (\lambda_{trans,z} - 0{,}2) + \lambda_{trans,z}^2) = 1{,}979$$

$$\chi_z = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans,z}^2}} = 0{,}308$$

In Y-Y Axis:

$$N_{cr,y} = \pi^2 * E * \frac{I_y}{(H * 100)^2} = 60663{,}06 \text{ kN}$$

$$\lambda_{trans,y} = \sqrt{\beta_A * A * \frac{f_y}{N_{cr,y}}} = 0{,}40$$

Apply strut curve c

$$\alpha = 0{,}49$$

$$\phi = 0{,}5 * (1 + \alpha * (\lambda_{trans,y} - 0{,}2) + \lambda_{trans,y}^2) = 0{,}629$$

$$\chi_y = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans,y}^2}} = 0{,}897$$

$$\chi = \mathrm{MIN}(\chi_z; \chi_y) = 0{,}3080$$

$$N_{b,Rd} = \chi * A_{eff} * \frac{f_y}{\gamma_M} = 2667{,}53 \text{ kN}$$

Limit bending moment about the Y-Y axis:

Cross-section class 4

$$z'_s = \frac{-b_{eff} * t_2 * \frac{h+t_2}{2} + \left(b + \frac{t_1}{2}\right) * t_2 * \frac{h+t_2}{2}}{A_{eff}} = 0{,}382 \text{ cm}$$

$$\frac{(h+t_2)^3}{12} * t_1 + (h+t_2) * t_1 * z'_s = 2261{,}46 \text{ cm}^4$$

$$b_{eff} * t_2 * \left(\frac{h+t_2}{2} + z'_s\right)^2 + \left(b + \frac{t_1}{2}\right) * t_2 * \left(\frac{h+t_2}{2} - z'_s\right)^2 = 2457{,}57 \text{ cm}^4$$

$I_{eff,y} =$ **4719,03 cm^4**

$$W_{eff,y} = \frac{I_{eff,y}}{\frac{h+t_2}{2} + z'_s} = 306{,}79 \text{ cm}^3$$

$$M_{c,Rd,y} = W_{eff,y} * \frac{f_y}{\gamma_M} = 65541{,}50 \text{ kNm}$$

Limit bending moment about the Z-Z axis:

Cross-section class 4 Tab 5.3.3

$$\psi = \frac{-y_s}{b + \frac{t_1}{2} - y_s} = -0{,}17$$

$$k_s = 0{,}57 - 0{,}21 * \psi + 0{,}07 * \psi^2 = 0{,}61$$

$$\frac{b + \frac{t_1}{2}}{t_2 * 21 * \varepsilon * \sqrt{k_s}} = 1{,}02 > 1$$

As in Table:5.3.5 (3)

$$\bar{\lambda}_{trans,p} = \frac{b + \frac{t_1}{2}}{t_2 * 28{,}4 * \varepsilon * \sqrt{k_s}} = 0{,}75 > 0{,}673$$

$$\rho = \frac{\bar{\lambda}_{trans,p} - 0{,}22}{\bar{\lambda}_{trans,p}^2} = 0{,}942$$

$$b_{eff} = \rho * \frac{b + \frac{t_1}{2}}{1 - \psi} = 8{,}05 \text{ cm}$$

$$b_t = -1 * \left(b + \frac{t_1}{2}\right) * \frac{\psi}{1 - \psi} = 1{,}45 \text{ cm}$$

$$c_{eff} = b_{eff} + b_t = 9{,}50 \text{ cm}$$

$$A_{eff} = (h + t_2) * t_1 + 2 * c_{eff} * t_2 \quad = \quad 41{,}40\ cm^2$$

$$y' = 2 * c_{eff}^{\ 2} * \frac{t_2}{2 * A_{eff}} \quad = \quad 1{,}31\ cm$$

$$I_{eff,z} = (h + t_2) * t_1 * y'^2 + 2 * \frac{c_{eff}^{\ 3}}{12} * t_2 + 2 * c_{eff} * t_2 * \left(\frac{c_{eff}}{2} - y'\right)^2 \quad = \quad 272{,}12\ cm^4$$

$$W_{eff,z} = \frac{I_{eff,z}}{\frac{c_{eff}}{2} - y'} \quad = \quad 79{,}10\ cm^3$$

$$M_{c,Rd,z} = W_{eff,z} * \frac{f_y}{\gamma_M} \quad = \quad 16898{,}64\ kNm$$

Check buckling:

$$N_{Ed} = N_d \quad = \quad 90{,}00\ kN$$

$$M_{y,Ed} = q_d * \frac{H^2}{8} \quad = \quad 30{,}00\ kNm$$

$$M_{z,Ed} = N_d * \frac{e_{Nz}}{100} \quad = \quad 0{,}30\ kNm$$

$$\beta_{My} = \quad 1{,}30$$

$$\mu_y = \bar{\lambda}_{trans,y} * (2 * \beta_{My} - 4) \quad = \quad -0{,}560 < 0{,}9$$

$$k_y = 1 - \frac{\mu_y * N_{Ed}}{\chi_y * A_{eff} * f_y} \quad = \quad 1{,}006 < 1{,}5$$

$$\psi = \quad 1{,}000$$

$$\beta_{My} = 1{,}8 - 0{,}7 * \psi \quad = \quad 1{,}10 < 1{,}5$$

$$\mu_z = \bar{\lambda}_{trans,z} * (2 * \beta_{My} - 4) \quad = \quad -2{,}736 < 0{,}9$$

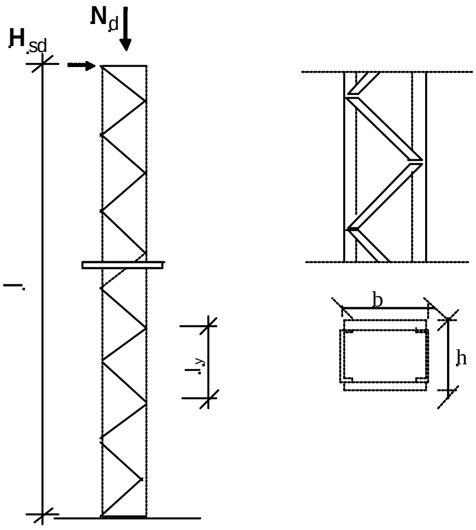
$$k_{z1} = 1 - \frac{\mu_z * N_{Ed}}{\chi * A_{eff} * f_y} \quad = \quad 1{,}082$$

$$k_{z2} = \quad 1{,}500$$

$$k_z = MIN(k_{z1}; k_{z2}) \quad = \quad 1{,}082$$

$$\frac{N_{Ed}}{\chi * A_{eff} * \frac{f_y}{\gamma_M}} + k_y * 100 * \frac{M_{y,Ed}}{W_{eff,y} * \frac{f_y}{\gamma_M}} + k_z * 100 * \frac{M_{z,Ed}}{W_{eff,z} * \frac{f_y}{\gamma_M}} \quad = \quad \underline{0{,}081 < 1}$$

Laced column made up of channel and angle sections:



Loads:

N_d =			300,00 kN
H_d =			45,00 kN

Plan and elevation values:

Span length l_y =			69,20 cm
Column height l =	$10*l_y/100$	=	6,92 m
Width b =			40,00 cm
Winkel j_1 =			45,00 °
U-Profil U =	SEL("steel/U"; Name;)	=	U 300
A_f =	TAB("steel/U"; A; Name=U)	=	58,80 cm²
e_z =	TAB("steel/U"; ez; Name=U)	=	2,70 cm
i_z =	TAB("steel/U"; iz; Name=U)	=	2,90 cm
Winkel W =	SEL("steel/WG"; Name;)	=	L 50x5
Angle thickness A_D =	TAB("steel/WG"; A; Name=W)	=	4,80 cm²
Angle thickness i_z =	TAB("steel/WG"; iz; Name=W)	=	0,98 cm
steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
E =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
ε =	$\sqrt{\frac{235}{f_y}}$	=	1,00
l_1 =	93,90 * ε	=	93,90

Partial safety factors:

g_M =	1,10

Maximal tension force in chord:

Effective length ratio k = 2,00

Imperfection:

$w_o = \frac{2 \cdot l}{5}$ = 2,77 cm

Effective moment of inertia:

$h_o = b - 2 \cdot e_z$ = 34,60 cm

$I_{eff} = 0{,}5 \cdot h_o^2 \cdot A_f$ = 35196,50 cm^4

Shear strength:

$S_v = 2 \cdot \frac{E}{10} \cdot A_D \cdot l_y \cdot \frac{h_o^2}{2 \cdot (2 \cdot h_o^2)^{3/2}}$ = 71276,36 kN

$N_{cr} = \frac{1}{4 \cdot \frac{(100 \cdot l)^2}{\pi^2 \cdot \frac{E}{10} \cdot I_{eff}} + \frac{1}{S_v}}$ = 3615,26 kN

$max\_M_E = H_d \cdot l + \frac{N_d}{1 - \frac{N_d}{N_{cr}}} \cdot \frac{w_o}{100}$ = 320,46 kNm

$N_{f,Ed} = 0{,}5 \cdot N_d + 100 \cdot \frac{max\_M_E}{h_o}$ = 1076,18 kN

Analysis of channels:

$\lambda = \frac{l_y}{i_z}$ = 23,86

$\lambda_{trans} = \frac{\lambda}{\lambda_1 \cdot \varepsilon}$ = 0,254

Apply strut curve c

$\alpha =$ 0,49

$\phi = 0{,}5 \cdot (1 + \alpha \cdot (\lambda_{trans} - 0{,}2) + \lambda_{trans}^2)$ = 0,545

$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans}^2}}$ = 0,9735

$N_{b,Rd} = \chi \cdot A_f \cdot \frac{f_y}{\gamma_M \cdot 10}$ = 1222,89 kN

$\frac{N_{f,Ed}}{N_{b,Rd}}$ = **0,880 < 1**

Analysis of lacing angles:

$$max\_V_E = H_d + \frac{N_d}{1 - \frac{N_d}{N_{cr}}} * w_o * \frac{p}{200 * l} \qquad = \qquad 47{,}06 \text{ kN}$$

Design force for a diagonal:

$$N_{Ed} = \frac{max\_V_E}{2 * \cos(j_1)} \qquad = \qquad 33{,}28 \text{ kN}$$

$$l = \frac{\ddot{O}\,\overline{2 * h_o{}^2}}{i_z} \qquad = \qquad 49{,}93$$

$$l_{trans} = \frac{l}{l_1 * e} \qquad = \qquad 0{,}532$$

Apply strut curve c

$$a = \qquad 0{,}49$$

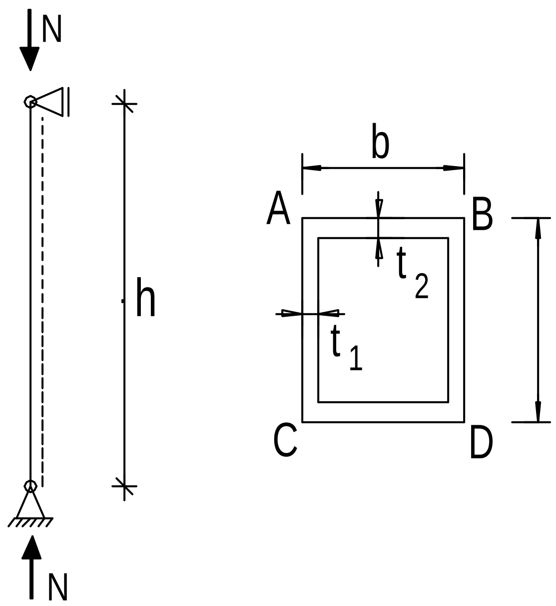
$$j = 0{,}5 * (1 + a * (l_{trans} - 0{,}2) + l_{trans}{}^2) \qquad = \qquad 0{,}723$$

$$c = \frac{1}{j + \ddot{O}\,\overline{j^2 - l_{trans}{}^2}} \qquad = \qquad 0{,}8247$$

$$N_{b,Rd} = c * A_D * \frac{f_y}{g_M * 10} \qquad = \qquad 84{,}57 \text{ kN}$$

$$\frac{N_{Ed}}{N_{b,Rd}} \qquad = \qquad \underline{\mathbf{0{,}39 < 1}}$$

Column of section class 4:



Load diagram:

Column height H =	6,50 m
Beam width b =	50,00 cm
Column height h =	100,00 cm
Web thickness t_1 =	1,00 cm
Flange thickness t_2 =	1,00 cm

Loads:

N_d = 3500,00 kN

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
E =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
ε =	$\sqrt{\frac{235}{f_y}}$	=	1,00

Partial safety factors:

γ_M = 1,10

γ_g = 1,35

Section classification:

As in Table 5.3.1

b' = $b - 2 * t_1$ = 48,00 cm

$\frac{b'}{t_2 * 42 * \varepsilon}$ = 1,14 > 1

Section class 4

h' = $h - 2 * t_2$ = 98,00 cm

$\frac{h'}{t_1 * 42 * \varepsilon}$ = 2,33 > 1

Section class 4

Effective web area:

Part A-B; C-D As in Table 5.3.2:

y = 1,00

k_s = 4,00

as in 5.3.5(3)

$$l_{quer,p} = \frac{b'}{t_2 * 28,4 * e * Ö\overline{k_s}} = 0,85 > 0,673$$

$$r = \frac{l_{quer,p} - 0,22}{l_{quer,p}^{2}} = 0,872$$

b_{eff} = r * b' = 41,86 cm

Part A-C; B-D As in Table 5.3.2:

y = 1,00 cm

k_s = 4,00

as in 5.3.5(3)

$$l_{quer,p} = \frac{h'}{t_1 * 28,4 * e * Ö\overline{k_s}} = 1,73 > 0,673$$

$$r = \frac{l_{quer,p} - 0,22}{l_{quer,p}^{2}} = 0,505$$

h_{eff} = r * h' = 49,49 cm

<u>Check buckling:</u>

$A_{eff} = 2 * (h_{eff} * t_1 + b_{eff} * t_2) + 4 * t_1 * t_2 \quad = \quad 186{,}70\ cm^2$

$A = b * h - (b - 2 * t_1) * (h - 2 * t_2) \quad = \quad 296{,}00\ cm^2$

$$\beta_A = \frac{A_{eff}}{A} \quad = \quad 0{,}631$$

$$I_b = \frac{b * h^3 - (b - 2 * t_1) * (h - 2 * t_2)^3}{12} \quad = \quad 401898{,}67\ cm^4$$

$$I_h = \frac{b^3 * h - (b - 2 * t_1)^3 * (h - 2 * t_2)}{12} \quad = \quad 138498{,}67\ cm^4$$

$I = MIN(I_b; I_h) \quad = \quad 138498{,}67\ cm^4$

$$N_{cr} = \pi^2 * \frac{E * I}{H^2 * 10^5} \quad = \quad 67941{,}82\ kN$$

$$\lambda_{transv} = \sqrt{\beta_A * A * \frac{f_y}{10 * N_{cr}}} \quad = \quad 0{,}25$$

Apply strut curve b

$\alpha = \quad 0{,}34$

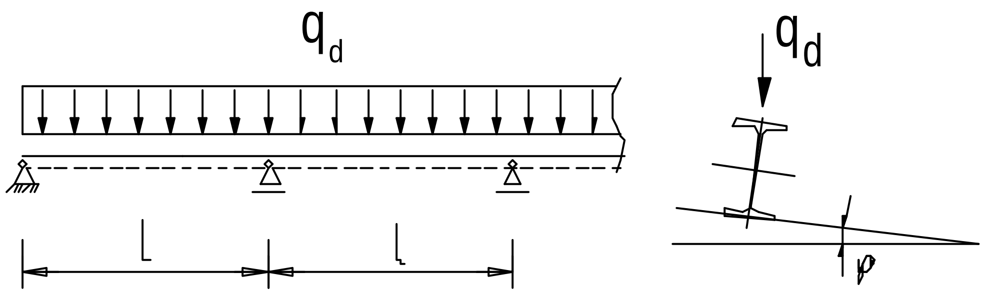
$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{transv} - 0{,}2) + \lambda_{transv}^2) \quad = \quad 0{,}540$

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{transv}^2}} \quad = \quad 0{,}9817$$

$$N_{b,Rd} = \chi * \beta_A * A * \frac{f_y}{\gamma_M * 10} \quad = \quad 3917{,}19\ kN$$

$$\frac{N_d}{N_{b,Rd}} \quad = \quad \underline{0{,}893 < 1}$$

Purlin / Strut with biaxial bending and torsional-flexural buckling:



Plan and elevation values:

Span length l=	720,00 cm
Angle of slope j =	10,00 °
Spacing of purlin l'=	4,00 m
platischer Formbeiwert a_{ply} =	1,14
platischer Formbeiwert a_{plz} =	1,25

Loads:

From dead load g=	0,20 kN/m²
From snow load s=	0,40 kN/m²

Section classification for: IPE 200

Profil Typ =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil =	SEL("steel/"Typ; Name;)	=	IPE 200
Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	200,00 mm
Flange width b_f =	TAB("steel/"Typ; b; Name=Profil)	=	100,00 mm
Flange thickness t_f =	TAB("steel/"Typ; t; Name=Profil)	=	8,50 mm
Flange thickness t_w =	TAB("steel/"Typ; s; Name=Profil)	=	5,60 mm
Moment of inertia I_y =	TAB("steel/"Typ; Iy; Name=Profil)	=	1940,00 cm⁴
Moment of inertia I_z =	TAB("steel/"Typ; Iz; Name=Profil)	=	142,00 cm⁴
Moment of inertia I_y =	TAB("steel/"Typ; IT; Name=Profil)	=	6,98 cm⁴
I_w =	$t_f * b_f^3 * (h - t_f)^2 / (24*10^6)$	=	12988,09 cm⁶
i_z =	TAB("steel/"Typ; iz; Name=Profil)	=	2,24 cm
W_{ely}=	TAB("steel/"Typ; Wy; Name=Profil)	=	194,00 cm³
W_{ply} =	$a_{ply} * W_{ely}$	=	221,16 cm³
W_{elz}=	TAB("steel/"Typ; Wz; Name=Profil)	=	28,50 cm³

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
G =	TAB("steel/EC"; G; Name=steel)	=	81000,00 N/mm²
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
ε =	$\sqrt{\frac{235}{f_y}}$	=	1,00

Partial safety factors:

g_M =	1,10
g_g =	1,35
g_p =	1,50

Design calculations:

Factored load p_d =	$\gamma_g * g + \gamma_p * s$	=	0,87 kN/m²
Effective load q_d =	$p_d * 4$	=	3,48 kN/m
Vertical load component q_{zd} =	$q_d * COS(\varphi)$	=	3,43 kN/m
Horizontal load component q_{yd} =	$q_d * SIN(\varphi)$	=	0,60 kN/m

Forces at 2nd support:

$$M_{byEd} = 0{,}105 * q_{zd} * \left(\frac{l}{100}\right)^2 = 18{,}67 \text{ kNm}$$

$$M_{bzEd} = 0{,}105 * q_{yd} * \left(\frac{l}{100}\right)^2 = 3{,}27 \text{ kNm}$$

$$V_{bzEd} = 0{,}606 * q_{zd} * \frac{l}{100} = 14{,}97 \text{ kN}$$

$$V_{byEd} = 0{,}606 * q_{yd} * \frac{l}{100} = 2{,}62 \text{ kN}$$

Check biaxial bending strength:

About Y-Y axis

$$A_v = 1{,}04 * h * t_w / 10^2 = 11{,}65 \text{ cm}^2$$

$$V_{pl,z,Rd} = A_v * \frac{f_y}{\sqrt{3} * \gamma_M * 10} = 143{,}69 \text{ kN}$$

$$\frac{V_{bzEd}}{V_{pl,z,Rd}} = \underline{0{,}10 < 1}$$

$$\frac{V_{bzEd}}{V_{pl,z,Rd}} = \underline{0{,}10 < 0{,}5}$$

⇒ No reduction in moment strength necessary due to shear.

$$M_{pl,y,Rd} = \frac{W_{ply} * f_y}{\gamma_M * 10} = 4724{,}78 \text{ kNcm}$$

About Z-Z axis

$$A_v = 2 * b_f * t_f / 10^2 = 17{,}00 \text{ cm}^2$$

$$V_{pl,y,Rd} = A_v * \frac{f_y}{\sqrt{3} * \gamma_M * 10} = 209{,}68 \text{ kN}$$

$$\frac{V_{byEd}}{V_{pl,y,Rd}} = \underline{0{,}01 < 1}$$

$$\frac{V_{byEd}}{V_{pl,y,Rd}} = \underline{0{,}01 < 0{,}5}$$

⇒ No reduction in moment strength necessary due to shear

$$M_{pl,z,Rd} = 1{,}5 * \frac{W_{elz} * f_y}{\gamma_M * 10} = 913{,}30 \text{ kNcm}$$

Analysis:

as in 5.35: for I- and H- sections:

$\alpha =$ 2,00

$\beta =$ 1,00

Or else: 5.4.8.1 (11)

$$\text{abs}\left(\left(\frac{M_{byEd} * 10^2}{M_{pl,y,Rd}}\right)^{\alpha} + \left(\frac{M_{bzEd} * 10^2}{M_{pl,z,Rd}}\right)^{\beta}\right) = \underline{0{,}514 < 1}$$

Check flexural-torsional buckling strength:

$$i_{LT} = \sqrt[4]{\frac{I_z * I_w}{W_{ply}^2}} = 2{,}48 \text{ cm}$$

Load is applied in top flange.

$z_a =$ 10,00 cm

$z_s =$ 0,00 cm

$z_g = z_a - z_s =$ 10,00 cm

As in Table F.1.2, interpolated.

$C_1 =$ 1,00

$C_2 =$ 0,80

$C_3 =$ 0,70

For simply supported ends – fixed ends k = 0,70

No measures taken toward bending $k_w =$ 1,00

$$h_s = \frac{h - t_f}{10} = 19{,}15 \text{ cm}$$

5.5.2; Section class 1+2 $\beta_w =$ 1,00

$$\lambda_1 = \pi * \sqrt{\frac{E_s}{f_y}} = 93{,}91$$

$$\lambda_{LT} = \frac{k * l / i_{LT}}{\sqrt{C_1} * \sqrt{\sqrt{\left(\frac{k}{k_w}\right)^2 + \frac{1}{20} * \left(\frac{k * l / i_{LT}}{\frac{h}{t_f}}\right)^2 + \left(\frac{2 * C_2 * z_g}{h_s}\right)^2} - \frac{2 * C_2 * z_g}{h_s}}} = 172{,}86$$

$$\lambda_{trans,LT} = \frac{\lambda_{LT}}{\lambda_1} * \sqrt{\beta_w} = 1{,}841$$

Apply strut curve a:

As in Table 5.5.1 $\alpha =$ 0,21

$$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans,LT} - 0{,}2) + \lambda_{trans,LT}^2) = 2{,}37$$

$$\chi_{LT} = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{trans,LT}^2}} = 0{,}26$$

No longitudinal forces $k_z =$ 1,00

No longitudinal forces $k_{LT} =$ 1,00

$$k_{LT} * \frac{M_{byEd}}{\chi_{LT} * \frac{M_{pl,y,Rd}}{100}} + k_z * \frac{M_{bzEd}}{\frac{M_{pl,z,Rd}}{100}} = \underline{\mathbf{1,88 < 1}}$$

Section is not adequate: Restrain section in quarterly points:

$l =$ l / 4 = 180,00 cm

$$\lambda_{LT} = \frac{k * l / i_{LT}}{\sqrt{C_1} * \sqrt{\sqrt{\left(\frac{k}{k_w}\right)^2 + \frac{1}{20} * \left(\frac{k * l / i_{LT}}{\frac{h}{t_f}}\right)^2 + \left(\frac{2 * C_2 * z_g}{h_s}\right)^2} - \frac{2 * C_2 * z_g}{h_s}}} = 85,08$$

$$\lambda_{trans,LT} = \frac{\lambda_{LT}}{\lambda_1} * \sqrt{\beta_w} = 0,906$$

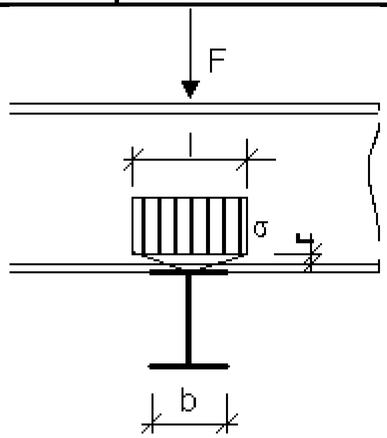
Apply strut curve a:

$$\varphi = 0,5 * (1 + \alpha * (\lambda_{trans,LT} - 0,2) + \lambda_{trans,LT}^2) = 0,98$$

$$\chi_{LT} = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{trans,LT}^2}} = 0,74$$

$$k_{LT} * \frac{M_{byEd}}{\chi_{LT} * \frac{M_{pl,y,Rd}}{100}} + k_z * \frac{M_{bzEd}}{\frac{M_{pl,z,Rd}}{100}} = \underline{\mathbf{0,89 < 1}}$$

Concentrated point load of a beam in another beam:



Design loads and properties

P_d =	68,00 kN
M_{Ed1}=	70,00 kNm
M_{Ed2}=	-22,00 kNm

Top beam (IPE180)

Profil TypO =	SEL("steel/Profils"; Name;)	=	IPE
Selected ProfilO =	SEL("steel/"TypO; Name;)	=	IPE 180
Web thickness s_o =	TAB("steel/"TypO; s; Name=ProfilO)	=	5,30 mm
Flange thickness t_o =	TAB("steel/"TypO; t; Name=ProfilO)	=	8,00 mm
Radius r_o =	TAB("steel/"TypO; r; Name=ProfilO)	=	9,00 mm
Flange width b_o =	TAB("steel/"TypO; b; Name=ProfilO)	=	91,00 mm
Web depth h_o=	TAB("steel/"TypO; h; Name=ProfilO)	=	180,00 mm
Web depth h_{1o}=	TAB("steel/"TypO; h1; Name=ProfilO)	=	146,00 mm
W_{elo} =	TAB("steel/"TypO; W_y; Name=ProfilO)	=	146,00 cm^3
W_{plo} =	1,14 * W_{elo}	=	166,44 cm^3
I_{yo}=	TAB("steel/"TypO; Iy; Name=ProfilO)	=	1320,00 cm^4

Bottom beam (HEA200)

Profil TypO =	SEL("steel/Profils"; Name;)	=	HEA
Selected ProfilO =	SEL("steel/"TypO; Name;)	=	HEA 200
Web thickness of beam s_u =	TAB("steel/"TypO; s; Name=ProfilO)	=	6,50 mm
Flange thickness t_u =	TAB("steel/"TypO; t; Name=ProfilO)	=	10,00 mm
Radius r_u =	TAB("steel/"TypO; r; Name=ProfilO)	=	18,00 mm
Flange width bu =	TAB("steel/"TypO; b; Name=ProfilO)	=	200,00 mm
Web depth h_u=	TAB("steel/"TypO; h; Name=ProfilO)	=	190,00 mm
Web depth h_{1u}=	TAB("steel/"TypO; h1; Name=ProfilO)	=	134,00 mm
W_{elu} =	TAB("steel/"TypO; W_y; Name=ProfilO)	=	389,00 cm^3
W_{plu} =	1,14 * W_{elu}	=	443,46 cm^3
I_{yu}=	TAB("steel/"TypO; Iy; Name=ProfilO)	=	3690,00 cm^4

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
G =	TAB("steel/EC"; G; Name=steel)	=	81000,00 N/mm²
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
e =	$Ö(235 / f_y)$	=	1,00
g_M =			1,10
l_1 =	93,90 * e	=	93,90
f_{yd}=	f_y/g_M	=	213,64 kN/cm²

Limit plastic bearing:

Stiff bearing length of top, bottom beam:

$s_{s1} = s_o + 2 * t_o + 4 * r_o * (1 - \frac{Ö\overline{2}}{2})$ = 31,84 mm

$s_{f,Ed1} = 100 * \frac{M_{Ed1}}{W_{elu}}$ = 17,99 kN/cm²

$b_{f1} = t_u * 25$ = 250,00 mm

b_{f1} = MIN(b_{f1}; b_u) = 200,00 mm

$s_{y1} = 2 * t_u * \ddot{O}\overline{\frac{b_u}{s_u}} * \ddot{O}\overline{1 - \left(\frac{g_M * s_{f,Ed1} * 10}{f_y}\right)^2}$ = 59,83 mm

$R_{y,Rd1} = (s_{s1} + s_{y1}) * s_u * \frac{f_{yd}}{10^3}$ = 127,30 kN

$\frac{P_d}{R_{y,Rd1}}$ = **0,53 < 1**

Stiff bearing length of bottom beam:

$s_{s2} = s_u + 2 * t_u + 4 * r_u * (1 - \frac{Ö\overline{2}}{2})$ = 47,59 mm

$s_{f,Ed2} = 100 * \frac{M_{Ed2}}{W_{elo}}$ = -15,07 kN/cm²

$b_f = t_o * 25$ = 200,00 mm

b_{f2} = MIN(b_f; b_o) = 91,00 mm

$s_{y2} = 2 * t_o * \ddot{O}\overline{\frac{b_o}{s_o}} * \ddot{O}\overline{1 - \left(\frac{g_M * s_{f,Ed2} * 10}{f_y}\right)^2}$ = 46,99 mm

$R_{y,Rd2} = (s_{s2} + s_{y2}) * s_o * \frac{f_{yd}}{10^3}$ = 107,09 kN

$\frac{P_d}{R_{y,Rd2}}$ = **0,63 < 1**

Check web crushing:

Bottom beam:

$m_1 = \dfrac{s_{s1}}{h_{1u}}$ = 0,238

$m_2 =$ 0,200

$m_u =$ MIN(m_1; m_2) = 0,200

$$R_{a,Rd1} = 0{,}5 * s_u^2 * \sqrt{E_s * f_y} * \frac{\sqrt{\dfrac{t_u}{s_u}} + 3 * \dfrac{s_u}{t_u} * m_u}{\gamma_M * 10^3} = 219{,}95 \text{ kN}$$

$$\frac{P_d}{R_{a,Rd1}} = \underline{\mathbf{0{,}31 < 1}}$$

$$M_{c,Rd1} = W_{plu} * \frac{f_{yd}}{10^3} = 94{,}74 \text{ kNm}$$

$$\frac{M_{Ed1}}{M_{c,Rd1}} = \underline{\mathbf{0{,}74 < 1}}$$

Top beam:

$m_1 = \dfrac{s_{s2}}{h_{1o}}$ = 0,326

$m_2 =$ 0,200

$m_o =$ MIN(m_1; m_2) = 0,200

$$R_{a,Rd2} = 0{,}5 * s_o^2 * \sqrt{E_s * f_y} * \frac{\sqrt{\dfrac{t_o}{s_o}} + 3 * \dfrac{s_o}{t_o} * m_o}{\gamma_M * 10^3} = 145{,}85 \text{ kN}$$

$$\frac{P_d}{R_{a,Rd2}} = \underline{\mathbf{0{,}47 < 1}}$$

$$M_{c,Rd2} = W_{plo} * \frac{f_{yd}}{10^3} = 35{,}56 \text{ kNm}$$

$$\frac{\text{abs}\,(M_{Ed2})}{M_{c,Rd2}} = \underline{\mathbf{0{,}62 < 1}}$$

Check web buckling for whole web:

Bottom beam:

Effective web width b_{eff2} = $\sqrt{h_u^2 + s_{s1}^2}$ = 192,65 mm

Effective web area A_2 = $b_{eff2} * \frac{s_u}{100}$ = 12,52 cm²

Moment of inertia I_2 = $\frac{\frac{b_{eff2}}{10} * \left(\frac{s_u}{10}\right)^3}{12}$ = 0,441 cm⁴

Radius of gyration i_2 = $\sqrt{\frac{I_2}{A_2}}$ = 0,188 cm

Effective slenderness λ_{trans2} = $\frac{h_u}{i_2 * \lambda_1 * 10}$ = 1,076

Apply strut curve c:

α = 0,49

ϕ = $0,5 * (1 + \alpha * (\lambda_{trans2} - 0,2) + \lambda_{trans2}^2)$ = 1,294

χ_2 = $\frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans2}^2}}$ = 0,4968

$R_{b,Rd2}$ = $\chi_2 * A_2 * \frac{f_{yd}}{10}$ = 132,88 kN

$\frac{P_d}{R_{b,Rd2}}$ = **0,51 < 1**

Top beam:

Effective web width b_{eff1} = $\sqrt{h_o^2 + s_{s1}^2}$ = 182,79 mm

Effective web area A_1 = $b_{eff1} * \frac{s_o}{100}$ = 9,69 cm²

Moment of inertia I_1 = $\frac{\frac{b_{eff1}}{10} * \left(\frac{s_o}{10}\right)^3}{12}$ = 0,227 cm⁴

Radius of gyration i_1 = $\sqrt{\frac{I_1}{A_1}}$ = 0,153 cm

Effective slenderness λ_{trans1} = $\frac{h_o}{i_1 * \lambda_1 * 10}$ = 1,253

Apply strut curve c:

α = 0,49

ϕ = $0,5 * (1 + \alpha * (\lambda_{trans1} - 0,2) + \lambda_{trans1}^2)$ = 1,543

χ_1 = $\frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans1}^2}}$ = 0,4093

$R_{b,Rd1} = c_1 * A_1 * \frac{f_{yd}}{10}$ = 84,73 kN

$\frac{P_d}{R_{b,Rd1}}$ = **0,80 < 1**

Check web yield at load points:
Bottom beam:

$s_{x,Ed2} = M_{Ed1} * 10^2 * \frac{\frac{h_u}{2} - t_u}{I_{yu}}$ = 161,25 kN/cm²

$s_{z,Ed2} = \frac{10^3 * P_d}{(s_{s1} + 2 * t_u) * s_u}$ = 201,80 kN/cm²

$\left(\frac{s_{x,Ed2}}{f_{yd}}\right)^2 + \left(\frac{s_{z,Ed2}}{f_{yd}}\right)^2 - \frac{s_{x,Ed2}}{f_{yd}} * \frac{s_{z,Ed2}}{f_{yd}}$ = **0,75 < 1**

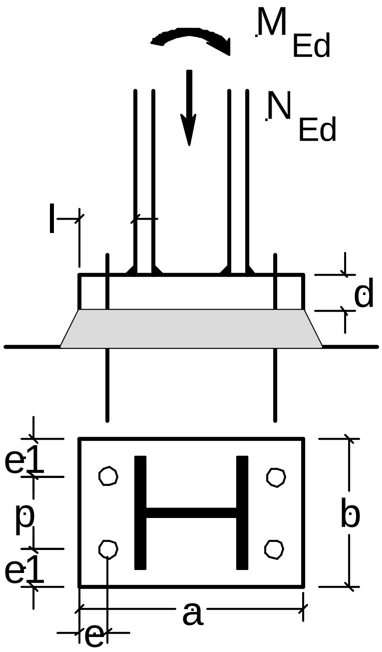
Top beam:

$s_{x,Ed1} = -M_{Ed2} * 10^2 * \frac{\frac{h_o}{2} - t_o}{I_{yo}}$ = 136,67 kN/cm²

$s_{z,Ed1} = \frac{10^3 * P_d}{(s_{s2} + 2 * t_o) * s_o}$ = 201,76 kN/cm²

$\left(\frac{s_{x,Ed1}}{f_{yd}}\right)^2 + \left(\frac{s_{z,Ed1}}{f_{yd}}\right)^2 - \frac{s_{x,Ed1}}{f_{yd}} * \frac{s_{z,Ed1}}{f_{yd}}$ = **0,70 < 1**

Column base subject to compression und bending:



Plan and elevation values:

Distance to edge of baseplate e =	45,00 mm
Distance to edge of baseplate e_1 =	100,00 mm
Spacing of bolts p =	300,00 mm
Width of baseplate b =	500,00 mm
Length of baseplate a =	500,00 mm
Plate thickness d =	30,00 mm
Projection of plate l =	100,00 mm
Weld thickness a_w =	10,00 mm
Projection of pad footing a_r =	1000,00 mm
Depth of pad footing h =	1400,00 mm
as in L.3 (idR) Bearing plane factor β_j = 2/3 =	0,67

Profil Typ =	SEL("steel/Profils"; Name;)	=	HEB
Selected Profil =	SEL("steel/"Typ; Name;)	=	HEB 300
Moment of inertia I_{y1} =	TAB("steel/"Typ; Iy; Name=Profil)	=	25170,00 cm^4
Flange width b_f =	TAB("steel/"Typ; b; Name=Profil)	=	300,00 mm
Flange thickness t_f =	TAB("steel/"Typ; t; Name=Profil)	=	19,00 mm
Column height h_t =	TAB("steel/"Typ; h; Name=Profil)	=	300,00 mm
$M_{pl,y,Rd}$ =	TAB("steel/"Typ; Mplyd; Name=Profil)	=	418,00 kNm
$N_{pl,Rd}$ =	TAB("steel/"Typ; Npld; Name=Profil)	=	3250,00 kN

Loads:

N_{Ed} =	1000,00 kN
M_{Ed} =	221,75 kNm

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 430
E_s =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
f_y =	TAB("steel/EC"; fy; Name=steel)	=	275,00 N/mm²
Concrete =	SEL("Concrete/EC"; Name;)	=	C30/37
f_{ck} =	TAB("Concrete/EC"; f_{ck}; Name=Concrete)	=	30,00 N/mm²
SC =	SEL("steel/bolt"; SC;)	=	4.6
Bolt =	SEL("steel/bolt"; BS;)	=	M 24
f_u =	TAB("steel/bolt"; fubk; SC=SC)	=	400,00 N/mm²
A_s =	TAB("steel/bolt"; Asp; BS=Bolt)	=	3,53 cm²

g_{Mb}= 1,25

g_c= 1,50

g_{M0}= 1,10

Design:

m =	$l - e - 0{,}8 * a_w * Ö\overline{2}$	=	43,69 mm
l_{eff1a} =	MIN(4 * m + 1,25 * e; 2 * m + 0,625 * e + e_1)	=	215,51 mm
l_{eff1b} =	MIN(2 * m + 0,625 * e + 0,5 * p; e_1 + 0,5 * p; b / 2)	=	250,00 mm
l_{eff1} =	MIN(l_{eff1a}; l_{eff1b})	=	215,51 mm
l_{eff2a} =	MIN(π * 2 * m; π * m + 2 * e_1)	=	274,51 mm
l_{eff2b} =	MIN(π * m + p; p + 2 * e_1)	=	437,26 mm
l_{eff} =	MIN(l_{eff2a}; l_{eff2b}; l_{eff1})	=	**215,51 mm**

Beanspruchbarkeit der Lagerplatte auf der Seite der Zugstangen:

$$M_{pl,1,Rd} = \frac{1}{4} * l_{eff} * \left(\frac{d}{10}\right)^2 * \frac{f_y}{g_{M0} * 10} = 12{,}12*10^3 \text{ kNmm}$$

$$n = e = 45{,}00 \text{ mm}$$

$$\frac{n}{1{,}25 * m} = 0{,}82 < 1$$

$$B_{t,Rd} = 0{,}9 * f_u * \frac{A_s}{g_{Mb} * 10} = 101{,}66 \text{ kN}$$

Full flange yield:

$$F_{T,Rd1} = 4 * \frac{M_{pl,1,Rd}}{m} = 1109{,}64 \text{ kN}$$

Bolt failure with flange yielding:

$$F_{T,Rd2} = \frac{2 * M_{pl,1,Rd} + n * 2 * B_{t,Rd}}{m + n} = 376{,}47 \text{ kN}$$

Bolt failure without flange yielding:

$$F_{T,Rd3} = 2 * B_{t,Rd} = 203{,}32 \text{ kN}$$

$$F_{T,Rd} = \text{MIN}(F_{T,Rd1}; F_{T,Rd2}; F_{T,Rd3}) = \mathbf{203{,}32 \text{ kN}}$$

Effective compression area:

Limit pressure in the base plate:

$a_1 =$ MIN(a + 2 * a_r; 5 * a; a + h) = 1900,00 mm

$k_j = \sqrt{\left(\frac{a_1^2}{a * b}\right)}$ = 3,80

$f_j = \beta_j * k_j * \frac{f_{ck}}{\gamma_c}$ = 50,92 N/mm²

$A_{eff} = 10 * \frac{N_{Ed} + F_{T,Rd}}{f_j}$ = 236,32 cm²

$c = d * \sqrt{\frac{f_y * 10}{3 * f_j * \gamma_{M0}}}$ = 121,36 mm

$x_0 = 100 * \frac{A_{eff}}{2 * c + b_f}$ = 43,54 mm

$\frac{x_0}{t_f + 2 * c}$ = **0,17 < 1**

Limit moment at column base:

$r_c = \frac{h_t}{2} + c - \frac{x_0}{2}$ = 249,59 mm

$M_{Rd} = \frac{F_{T,Rd} * 10^3 * \left(\frac{h_t}{2} + 100 - e\right) + A_{eff} * 100 * f_j * r_c}{10^6}$ = 342,02 kNm

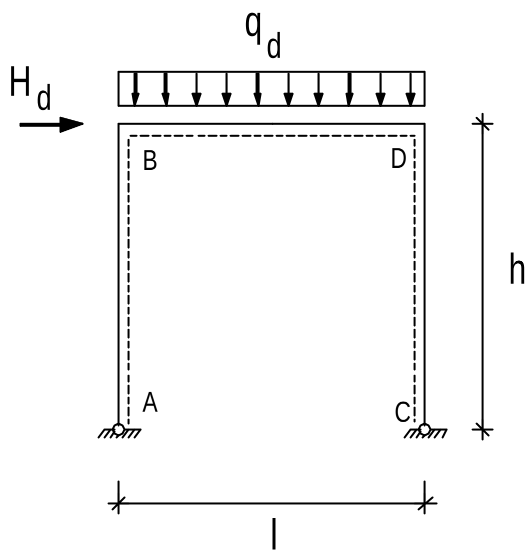
Check permissible strength of column to bending and compression.:

$M_{Ny,Rd} = 1,1 * M_{pl,y,Rd} * \left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right)$ = 318,32 kNm

Analysis:

$\mathrm{MAX}\left(\frac{M_{Ed}}{M_{Ny,Rd}}; \frac{M_{Ed}}{M_{Rd}}\right)$ = **0,70 < 1**

Sway frame with bracing out of plane of frame, plastic theory:



Plan and elevation values:

Frame width l =	8,00 m
Frame depth h =	6,00 m
Number of column n_c =	2
Number of storeys n_s =	1

Beam:

Profil Typ1 =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil1=	SEL("steel/"Typ1; Name;)	=	IPE 240
Column height h_1 =	TAB("steel/"Typ1; h; Name=Profil1)	=	240,00 mm
Web thickness s_1 =	TAB("steel/"Typ1; s; Name=Profil1)	=	6,20 mm
Moment of inertia I_{y1} =	TAB("steel/"Typ1; Iy; Name=Profil1)	=	3890,00 cm^4
Cross-sectional area A_1=	TAB("steel/"Typ1; A; Name=Profil1)	=	39,10 cm^2
Moment of resistance W_{y1}=	TAB("steel/"Typ1; Wy; Name=Profil1)	=	324,00 cm^2
Moment of resistance Wpl_{y1}=	1,14 * W_{y1}	=	369,36 cm^3

Column section:

Profil Typ2 =	SEL("steel/Profils"; Name;)	=	HEB
Selected Profil2 =	SEL("steel/"Typ2; Name;)	=	HEB 240
Flange width b_2 =	TAB("steel/"Typ2; b; Name=Profil2)	=	240,00 mm
Column height h_2 =	TAB("steel/"Typ2; h; Name=Profil2)	=	240,00 mm
Web thickness s_2 =	TAB("steel/"Typ2; s; Name=Profil2)	=	10,00 mm
Moment of inertia I_{y2} =	TAB("steel/"Typ2; Iy; Name=Profil2)	=	11260,00 cm^4
Moment of inertia I_{z2} =	TAB("steel/"Typ2; Iz; Name=Profil2)	=	3920,00 cm^4
Moment of inertia I_{t2} =	TAB("steel/"Typ2; IT; Name=Profil2)	=	103,00 cm^4
Moment of inertia I_w =	10^3*TAB("steel/"Typ2; Iw; Name=Profil2)	=	486900,00 cm^6
Cross-sectional area A_2=	TAB("steel/"Typ2; A; Name=Profil2)	=	106,000 cm^2
i_y =	TAB("steel/"Typ2; iy; Name=Profil2)	=	10,30 cm
i_z =	TAB("steel/"Typ2; iz; Name=Profil2)	=	6,08 cm
Moment of resistance W_{ely2} =	TAB("steel/"Typ2;Wy; Name=Profil2)	=	938,00 cm^3
Moment of resistance W_{ply2} =	1,25 * W_{ely2}	=	1172,50 cm^3

Loads:

H_d =	9,00 kN
q_d =	9,00 kN/m

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
G =	TAB("steel/EC"; G; Name=steel)	=	81000,00 N/mm²
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
e =	Ö(235 / f_y)	=	1,00
g_M =			1,10
l_1 =	93,90 * e	=	93,90

Frame classification as in 5.2.5.2.:

Frame sways

$$k = \frac{I_{y1}}{I_{y2}} * \frac{h}{l} = 0,259$$

Deflection at top of frame due to horizontal load:

H = 1,00 kN

$$d = \frac{(100 * h)^3 * 10}{12 * E_s * I_{y2}} * \frac{2 * k + 1}{k * H} = 0,446 \text{ cm}$$

Sumtotal of vertical loads:

V = q_d * l = 72,00 kN

Bracing is restrained out of plane of frame

$$\frac{d}{h * 100} * \frac{V}{H} = \underline{0,054 < 0,1}$$

Þ Sway frame, plastic theory, restrained from moving out of plane.

Design with plastic theory

Imperfection:

$$j_0 = \frac{1}{200} = 0,005$$

$$k_c = \text{MIN}(\sqrt{0,5 + \frac{1}{n_c}}; 1) = 1,00$$

$$k_s = \text{MIN}(\sqrt{0,2 + \frac{1}{n_s}}; 1) = 1,00$$

j =	$j_0 * k_c * k_s$	=	0,0050
DH_d =	j * V	=	0,36 kN
H_{Ed} =	H_d + DH_d	=	9,36 kN

Design calculations:

Reactions at column bases:

$H_l = \frac{q_d * l^2}{4 * h * (2 * k + 3)}$ = 6,82 kN

$V_l = q_d * \frac{l}{2}$ = 36,00 kN

$M_{Bl} = -H_l * h$ = -40,92 kNm

$H_2 = H_{Ed} / 2$ = 4,68 kN

$V_2 = H_{Ed} * \frac{h}{l}$ = 7,02 kN

$M_{B2} = H_2 * h$ = 28,08 kNm

Analysis of column

$N_{Ed} = V_l + V_2$ = 43,02 kN

$V_{Ed} = H_l + H_2$ = 11,50 kN

$M_D = M_{Bl} - M_{B2}$ = -69,00 kNm

$M_C =$ 0,00 kNm

Buckling in the plane of frame as in section E:

$k_c = \frac{I_{y2}}{h * 100}$ = 18,77 cm³

$k_1 = 1{,}5 * \frac{I_{y1}}{l * 100}$ = 7,29 cm³

$k_2 =$ 0,0

$h_2 =$ 1 (Hinge)

$h_1 = \frac{k_c}{k_c + k_1 + k_2}$ = 0,72

Slenderness ratio as in Fig. E.2.1.:

$l = 3{,}2 * h$ = 19,20 m

$l_y = 100 * \frac{l}{i_y}$ = 186,41

$l_{trans,y} = \frac{l_y}{l_1 * e}$ = 1,99

$\frac{h_2}{b_2}$ = 1,00 < 1,2

Apply strut curve b:

$a =$ 0,34

$b_A =$ 1,000

$j = 0{,}5 * (1 + a * (l_{trans,y} - 0{,}2) + l_{trans,y}^2)$ = 2,784

$c_y = \frac{1}{j + \sqrt{j^2 - l_{trans,y}^2}}$ = 0,2114

Buckling outside the plane of frame

$$\lambda_z = 100 * \frac{h}{i_z} = 98{,}68$$

$$\lambda_{trans,z} = \frac{\lambda_z}{\lambda_1 * \varepsilon} = 1{,}05$$

Apply strut curve c:

$\alpha =$ 0,49

$\beta_A =$ 1,000

$$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans,z} - 0{,}2) + \lambda_{trans,z}^2) = 1{,}260$$

$$\chi_z = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{trans,z}^2}} = 0{,}5111$$

$$\chi_{min} = MIN(\chi_y; \chi_z) = 0{,}2114$$

Check bending and buckling:

$\psi =$ 0,00

$$\beta_{My} = 1{,}8 - 0{,}7 * \psi = 1{,}80$$

$$\mu_y = \lambda_{trans,y} * (2 * \beta_{My} - 4) + \frac{W_{ply2} - W_{ely2}}{W_{ely2}} = -0{,}546 < 0{,}9$$

$$k_y = 1 - \frac{\mu_y * N_{Ed}}{\chi_y * A_2 * f_y} = 1{,}004 < 1{,}5$$

$$\frac{N_{Ed}}{\chi_{min} * A_2 * \frac{f_y}{\gamma_M * 10}} + 100 * k_y * \frac{abs(M_D)}{W_{ply2} * \frac{f_y}{\gamma_M * 10}} = \underline{\mathbf{0{,}37 < 1}}$$

Check torsional-flexural buckling:

$C_1 =$ 1,879

$$M_{cr} = C_1 * \frac{\pi^2 * E_s * I_{z2}}{(h * 100)^2 * 10} * \sqrt{\frac{I_w}{I_{z2}} + \frac{(h * 100)^2 * G * I_{t2}}{\pi^2 * E_s * I_{z2}}} = 94241{,}31 \text{ kNm}$$

As in Table: F.1.1

$\psi =$ 0,00

$\beta_W =$ 1,00

$k =$ 1,00

$$\lambda_{LT} = \sqrt{\pi^2 * \frac{E_s}{10} * \frac{W_{ply2}}{M_{cr}}} = 50{,}78$$

$$\lambda_{trans,LT} = \frac{\lambda_{LT}}{\lambda_1} * \sqrt{\beta_w} = 0{,}541$$

Apply strut curve a

$\alpha =$ 0,21

$$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans,LT} - 0{,}2) + \lambda_{trans,LT}^2) = 0{,}682$$

$$c_{LT} = \frac{1}{j + \text{Ö}\overline{j^{2} - l_{trans,LT}^{2}}} \qquad = \qquad 0{,}911$$

$$b_{M,LT} = 1{,}8 - 0{,}7 * y \qquad = \qquad 1{,}80$$

$$m_{LT} = 0{,}15 * l_{trans,z} * b_{M,LT} - 0{,}15 \qquad = \qquad 0{,}134 < 0{,}9$$

$$k_{LT} = 1 - \frac{m_{LT} * N_{Ed}}{c_{LT} * A_2 * f_y} \qquad = \qquad 1{,}000 < 1{,}5$$

$$\frac{N_{Ed}}{c_z * A_2 * \frac{f_y}{g_M * 10}} + 100 * k_{LT} * \frac{abs(M_D)}{c_{LT} * W_{ply2} * \frac{f_y}{g_M * 10}} \qquad = \qquad \underline{\mathbf{0{,}340 < 1}}$$

Section analysis at D:

Column:

$$A_v = 1{,}04 * h_2 * s_2 / 10^2 \qquad = \qquad 24{,}96 \text{ cm}^2$$

$$V_{pl,Rd} = A_v * \frac{f_y}{\text{Ö}\overline{3} * g_M * 10} \qquad = \qquad 307{,}86 \text{ kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} \qquad = \qquad \underline{\mathbf{0{,}037 < 0{,}5}}$$

No interaction

$$N_{pl,Rd} = A_2 * \frac{f_y}{g_M * 10} \qquad = \qquad 2264{,}55 \text{ kN}$$

$$n = \frac{N_{Ed}}{N_{pl,Rd}} \qquad = \qquad 0{,}02$$

$$M_{pl,y,Rd} = W_{ply2} * \frac{f_y}{g_M * 10^3} \qquad = \qquad 250{,}49 \text{ kNm}$$

$$M_{Ny,Rd} = 1{,}11 * M_{pl,y,Rd} * (1 - n) \qquad = \qquad 272{,}48 \text{ kNm}$$

$$MAX\left(\frac{abs(M_D)}{M_{pl,y,Rd}} ; \frac{abs(M_D)}{M_{Ny,Rd}}\right) \qquad = \qquad \underline{\mathbf{0{,}28 < 1}}$$

Beam:

$A_v =$ $1{,}04 * h_1 * s_1 / 10^2$ $=$ 15,48 cm²

$V_{pl,Rd} =$ $A_v * \dfrac{f_y}{\sqrt{3} * \gamma_M * 10}$ $=$ 190,93 kN

$\dfrac{N_{Ed}}{V_{pl,Rd}}$ $=$ **<u>0,225 < 0,5</u>**

No interaction

$N_{pl,Rd} =$ $A_1 * f_y / 10 / \gamma_M$ $=$ 835,32 kN

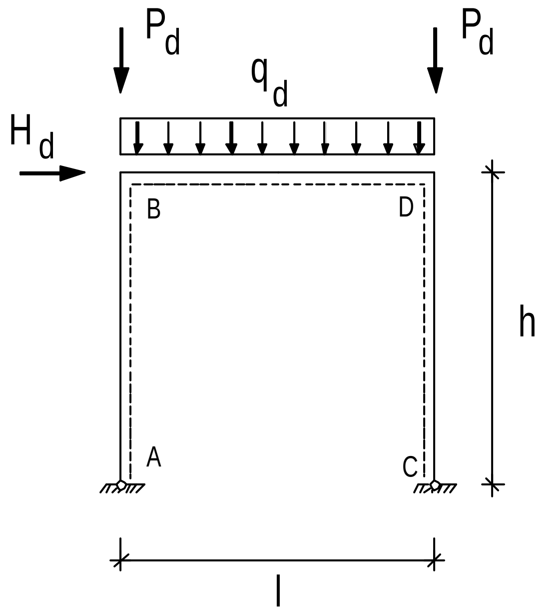
$n =$ $V_{Ed} / N_{pl,Rd}$ $=$ 0,014

$M_{pl,y,Rd} =$ $W_{ply1} * \dfrac{f_y}{\gamma_M * 10^3}$ $=$ 78,91 kNm

$M_{Ny,Rd} =$ $1{,}11 * M_{pl,y,Rd} * (1 - n)$ $=$ 86,36 kNm

$MAX\left(\dfrac{abs(M_D)}{M_{pl,y,Rd}} ; \dfrac{abs(M_D)}{M_{Ny,Rd}}\right)$ $=$ **<u>0,87 < 1</u>**

Frame, plastic with magnifying factor



Plan and elevation values:

Frame width l=	8,00 m
Frame depth h=	6,00 m
Number of column n_c=	2
Number of storeys n_s=	1

Beam:

Profil Typ1 =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil1=	SEL("steel/"Typ1; Name;)	=	IPE 240
Column height h_1 =	TAB("steel/"Typ1; h; Name=Profil1)	=	240,00 mm
Flange thickness s_1 =	TAB("steel/"Typ1; s; Name=Profil1)	=	6,20 mm
Moment of inertia I_{y1} =	TAB("steel/"Typ1; Iy; Name=Profil1)	=	3890,00 cm^4
Cross-sectional area A_1=	TAB("steel/"Typ1; A; Name=Profil1)	=	39,10 cm^2
Moment of resistance W_{y1}=	TAB("steel/"Typ1; Wy; Name=Profil1)	=	324,00 cm^2
Moment of resistance Wpl_{y1}=	1,14 * W_{y1}	=	369,36 cm^3

Column section:

Profil Typ2 =	SEL("steel/Profils"; Name;)	=	HEB
Selected Profil2 =	SEL("steel/"Typ2; Name;)	=	HEB 240
Flange width b_2 =	TAB("steel/"Typ2; b; Name=Profil2)	=	240,00 mm
Column height h_2 =	TAB("steel/"Typ2; h; Name=Profil2)	=	240,00 mm
Flange thickness s_2 =	TAB("steel/"Typ2; s; Name=Profil2)	=	10,00 mm
Moment of inertia I_{y2} =	TAB("steel/"Typ2; Iy; Name=Profil2)	=	11260,00 cm^4
Moment of inertia I_{z2} =	TAB("steel/"Typ2; Iz; Name=Profil2)	=	3920,00 cm^4
Moment of inertia I_{t2} =	TAB("steel/"Typ2; IT; Name=Profil2)	=	103,00 cm^4
Moment of inertia I_w =	10^3*TAB("steel/"Typ2; Iw; Name=Profil2)	=	486900,00 cm^6
Cross-sectional area A_2=	TAB("steel/"Typ2; A; Name=Profil2)	=	106,000 cm^2
i_y =	TAB("steel/"Typ2; iy; Name=Profil2)	=	10,30 cm
i_z =	TAB("steel/"Typ2; iz; Name=Profil2)	=	6,08 cm
Moment of resistance W_{ely2} =	TAB("steel/"Typ2;Wy; Name=Profil2)	=	938,00 cm^3
Moment of resistance W_{ply2} =	1,25 * W_{ely2}	=	1172,50 cm^3

Loads:

H_d = 9,00 kN

q_d = 8,00 kN/m

P_d= 100,00 kN

Materials and stresses:

steel = SEL("steel/EC"; Name;) = Fe 360

E_s = TAB("steel/EC"; E; Name=steel) = 210000,00 N/mm²

G = TAB("steel/EC"; G; Name=steel) = 81000,00 N/mm²

f_y = TAB("steel/EC"; fy; Name=steel) = 235,00 N/mm²

$\varepsilon = \sqrt{\frac{235}{f_y}}$ = 1,00

γ_M = 1,10

λ_1 = 93,90 * ε = 93,90

Frame classification as in 5.2.5.2.:

Frame sways

Classification of stiffness:

$k = \frac{I_{y1}}{I_{y2}} * \frac{h}{l}$ = 0,259

Deflection at top of frame due to horizontal load:

H = 1,00 kN

$\delta = \frac{(100 * h)^3 * 10}{12 * E_s * I_{y2}} * \frac{2 * k + 1}{k * H}$ = 0,446 cm

Sumtotal of vertical loads:

V = P_d * 2 + q_d * l = 264,00 kN

$\frac{\delta}{h * 100} * \frac{V}{H}$ = 0,1962 < 0,1

⇒ Frame, laterally unrestrained!;restrained from moving out of plane. mit Vergrößerungsfaktor!

Design with plastic theory and magnifying factor

Imperfection:

$\phi_0 = \frac{1}{200}$ = 0,005

k_c = MIN($\sqrt{0,5 + \frac{1}{n_c}}$; 1) = 1,00

k_s = MIN($\sqrt{0,2 + \frac{1}{n_s}}$; 1) = 1,00

ϕ = $\phi_0 * k_c * k_s$ = 0,0050

ΔH_d = ϕ * V = 1,32 kN

H_{Ed} = $H_d + \Delta H_d$ = 10,32 kN

Design calculations:

Reactions at column bases:

$H_l = \dfrac{q_d * l^2}{4 * h * (2 * k + 3)}$ = 6,06 kN

$V_l = q_d * \dfrac{l}{2}$ = 32,00 kN

$M_{Bl} = -H_l * h$ = -36,36 kNm

$H_2 = H_{Ed} / 2$ = 5,16 kN

$V_2 = H_{Ed} * \dfrac{h}{l}$ = 7,74 kN

$M_{B2} = H_2 * h$ = 30,96 kNm

Analysis of column

Dischinger factor:

$D = \dfrac{1}{1 - \dfrac{d}{h * 100} * \dfrac{V}{H}}$ = 1,244

$M_D = M_{Bl} - D * M_{B2}$ = -74,87 kNm

$N_{Ed} = V_l + D * V_2 + P_d$ = 141,63 kN

$V_{Ed} = H_l + D * H_2$ = 12,48 kN

Buckling in the plane of frame:

$k_c = \dfrac{I_{y2}}{h * 100}$ = 18,77 cm³

$k_1 = \dfrac{I_{y1}}{l * 100}$ = 4,86 cm³

$k_2 =$ 0,0

$h_2 =$ 1 (Hinge)

$h_1 = \dfrac{k_c}{k_c + k_1 + k_2}$ = 0,79

Slenderness ratio as in Fig. E.2.1.:

$l = 0{,}92 * h$ = 5,52 m

$l_y = 100 * \dfrac{l}{i_y}$ = 53,59

$l_{trans,y} = \dfrac{l_y}{l_1 * e}$ = 0,57

$\dfrac{h_2}{b_2}$ = 1,00 < 1,2

Apply strut curve b:

$a =$ 0,34

$b_A =$ 1,000

$j = 0{,}5 * (1 + a * (l_{trans,y} - 0{,}2) + l_{trans,y}^2)$ = 0,725

$$c_y = \frac{1}{j + \ddot{O}\overline{j^2 - l_{trans,y}^2}} = 0{,}8525$$

Buckling outside the plane of frame

$$l_z = 100 * \frac{h}{i_z} = 98{,}68$$

$$l_{trans,z} = \frac{l_z}{l_1 * e} = 1{,}05$$

Apply strut curve c:

$a =$ 0,49

$b_A =$ 1,000

$$j = 0{,}5 * (1 + a * (l_{trans,z} - 0{,}2) + l_{trans,z}^2) = 1{,}260$$

$$c_z = \frac{1}{j + \ddot{O}\overline{j^2 - l_{trans,z}^2}} = 0{,}5111$$

$$c_{min} = MIN(c_{y;}\ c_{z)} = 0{,}5111$$

Check bending and buckling:

$y =$ 0,00

$$b_{My} = 1{,}8 - 0{,}7 * y = 1{,}80$$

$$m_y = l_{trans,y} * (2 * b_{My} - 4) + \frac{W_{ply2} - W_{ely2}}{W_{ely2}} = 0{,}022 < 0{,}9$$

$$k_y = 1 - \frac{m_y * N_{Ed}}{c_y * A_2 * f_y} = 1{,}000 < 1{,}5$$

$$\frac{N_{Ed}}{c_{min} * A_2 * \frac{f_y}{g_M * 10}} + 100 * k_y * \frac{abs(M_D)}{W_{ply2} * \frac{f_y}{g_M * 10}} = \underline{\mathbf{0{,}42 < 1}}$$

Check torsional-flexural buckling:

$C_1 =$ 1,879

$$M_{cr} = C_1 * \frac{p^2 * E_s * I_{z2}}{(h * 100)^2 * 10} * \ddot{O}\overline{\frac{I_w}{I_{z2}} + \frac{(h * 100)^2 * G * I_{t2}}{p^2 * E_s * I_{z2}}} = 94241{,}31 \text{ kNm}$$

As in Table: F.1.1

$y =$ 0,00

$b_w =$ 1,00

$k =$ 1,00

$$l_{LT} = \ddot{O}\overline{p^2 * \frac{E_s}{10} * \frac{W_{ply2}}{M_{cr}}} = 50{,}78$$

$$l_{trans,LT} = \frac{l_{LT}}{l_1} * \ddot{O}\overline{b_w} = 0{,}541$$

Apply strut curve a

$a =$ 0,21

$j = 0{,}5 * (1 + a * (l_{trans,LT} - 0{,}2) + l_{trans,LT}^2) = 0{,}682$

$c_{LT} = \dfrac{1}{j + Ö\overline{j^2 - l_{trans,LT}^2}} = 0{,}911$

$b_{M,LT} = 1{,}8 - 0{,}7 * y = 1{,}80$

$m_{LT} = 0{,}15 * l_{trans,z} * b_{M,LT} - 0{,}15 = 0{,}134 < 0{,}9$

$k_{LT} = 1 - \dfrac{m_{LT} * N_{Ed}}{c_{LT} * A_2 * f_y} = 0{,}999 < 1{,}5$

$$\frac{N_{Ed}}{c_z * A_2 * \dfrac{f_y}{g_M * 10}} + 100 * k_{LT} * \frac{abs(M_D)}{c_{LT} * W_{ply2} * \dfrac{f_y}{g_M * 10}} = \underline{0{,}450 < 1}$$

Section analysis at D:
Column:

$A_v = 1{,}04 * h_2 * s_2 / 10^2 = 24{,}96\ cm^2$

$V_{pl,Rd} = A_v * \dfrac{f_y}{Ö\overline{3} * g_M * 10} = 307{,}86\ kN$

$\dfrac{V_{Ed}}{V_{pl,Rd}} = \underline{0{,}041 < 0{,}5}$

No interaction

$N_{pl,Rd} = A_2 * \dfrac{f_y}{g_M * 10} = 2264{,}55\ kN$

$n = \dfrac{N_{Ed}}{N_{pl,Rd}} = 0{,}063$

$M_{pl,y,Rd} = W_{ply2} * \dfrac{f_y}{g_M * 10^3} = 250{,}49\ kNm$

$M_{Ny,Rd} = 1{,}11 * M_{pl,y,Rd} * (1 - n) = 260{,}53\ kNm$

$MAX\left(\dfrac{abs(M_D)}{M_{pl,y,Rd}} ; \dfrac{abs(M_D)}{M_{Ny,Rd}}\right) = \underline{0{,}30 < 1}$

Beam:

$A_v =$ $1{,}04 * h_1 * s_1 / 10^2$ = 15,48 cm²

$V_{pl,Rd} =$ $A_v * \dfrac{f_y}{\sqrt{3} * \gamma_M * 10}$ = 190,93 kN

$\dfrac{N_{Ed}}{V_{pl,Rd}}$ = **0,742 < 0,5**

No interaction

$N_{pl,Rd} =$ $A_1 * f_y / 10 / \gamma_M$ = 835,32 kN

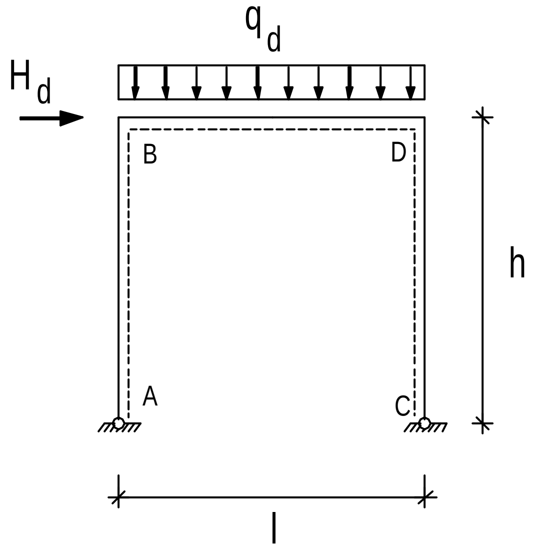
$n =$ $V_{Ed} / N_{pl,Rd}$ = 0,015

$M_{pl,y,Rd} =$ $W_{ply1} * \dfrac{f_y}{\gamma_M * 10^3}$ = 78,91 kNm

$M_{Ny,Rd} =$ $1{,}11 * M_{pl,y,Rd} * (1 - n)$ = 86,28 kNm

$\mathrm{MAX}\left(\dfrac{abs(M_D)}{M_{pl,y,Rd}} ; \dfrac{abs(M_D)}{M_{Ny,Rd}}\right)$ = **0,95 < 1**

Sway frame with bracing out of plane of frame, plastic theory:



Plan and elevation values:

Frame width l=	8,00 m
Frame depth h=	6,00 m
Number of column n_c=	2
Number of storeys n_s=	1

Beam:

Profil Typ1 =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil1=	SEL("steel/"Typ1; Name;)	=	IPE 240
Column height h_1 =	TAB("steel/"Typ1; h; Name=Profil1)	=	240,00 mm
Web thickness s_1 =	TAB("steel/"Typ1; s; Name=Profil1)	=	6,20 mm
Moment of inertia I_{y1} =	TAB("steel/"Typ1; Iy; Name=Profil1)	=	3890,00 cm^4
Cross-sectional area A_1=	TAB("steel/"Typ1; A; Name=Profil1)	=	39,10 cm^2
Moment of resistance W_{y1}=	TAB("steel/"Typ1; Wy; Name=Profil1)	=	324,00 cm^2
Moment of resistance Wpl_{y1}=	1,14 * W_{y1}	=	369,36 cm^3

Column section:

Profil Typ2 =	SEL("steel/Profils"; Name;)	=	HEB
Selected Profil2 =	SEL("steel/"Typ2; Name;)	=	HEB 240
Flange width b_2 =	TAB("steel/"Typ2; b; Name=Profil2)	=	240,00 mm
Column height h_2 =	TAB("steel/"Typ2; h; Name=Profil2)	=	240,00 mm
Web thickness s_2 =	TAB("steel/"Typ2; s; Name=Profil2)	=	10,00 mm
Moment of inertia I_{y2} =	TAB("steel/"Typ2; Iy; Name=Profil2)	=	11260,00 cm^4
Moment of inertia I_{z2} =	TAB("steel/"Typ2; Iz; Name=Profil2)	=	3920,00 cm^4
Moment of inertia I_{t2} =	TAB("steel/"Typ2; IT; Name=Profil2)	=	103,00 cm^4
Moment of inertia I_w =	10^3*TAB("steel/"Typ2; Iw; Name=Profil2)	=	486900,00 cm^6
Cross-sectional area A_2=	TAB("steel/"Typ2; A; Name=Profil2)	=	106,000 cm^2
i_y =	TAB("steel/"Typ2; iy; Name=Profil2)	=	10,30 cm
i_z =	TAB("steel/"Typ2; iz; Name=Profil2)	=	6,08 cm
Moment of resistance W_{ely2} =	TAB("steel/"Typ2;Wy; Name=Profil2)	=	938,00 cm^3
Moment of resistance W_{ply2} =	1,25 * W_{ely2}	=	1172,50 cm^3

Loads:

H_d = 9,00 kN

q_d = 9,00 kN/m

Materials and stresses:

steel = SEL("steel/EC"; Name;) = Fe 360

E_s = TAB("steel/EC"; E; Name=steel) = 210000,00 N/mm²

G = TAB("steel/EC"; G; Name=steel) = 81000,00 N/mm²

f_y = TAB("steel/EC"; fy; Name=steel) = 235,00 N/mm²

$\varepsilon = \sqrt{\dfrac{235}{f_y}}$ = 1,00

γ_M = 1,10

λ_1 = 93,90 * ε = 93,90

Frame classification as in 5.2.5.2.:

Frame sways

$k = \dfrac{I_{y1}}{I_{y2}} * \dfrac{h}{l}$ = 0,259

Deflection at top of frame due to horizontal load:

H = 1,00 kN

$\delta = \dfrac{(100 * h)^3 * 10}{12 * E_s * I_{y2}} * \dfrac{2 * k + 1}{k * H}$ = 0,446 cm

Sumtotal of vertical loads:

V = q_d * l = 72,00 kN

Bracing is restrained out of plane of frame

$\dfrac{\delta}{h * 100} * \dfrac{V}{H}$ = <u>**0,0535 < 0,1**</u>

⇒ Sway frame, plastic theory;restrained from moving out of plane.

Design with plastic theory and magnifying factor

Imperfection:

$\phi_0 = \dfrac{1}{200}$ = 0,005

k_c = MIN($\sqrt{0{,}5 + \dfrac{1}{n_c}}$; 1) = 1,00

k_s = MIN($\sqrt{0{,}2 + \dfrac{1}{n_s}}$; 1) = 1,00

ϕ = $\phi_0 * k_c * k_s$ = 0,0050

ΔH_d = ϕ * V = 0,36 kN

H_{Ed} = $H_d + \Delta H_d$ = 9,36 kN

Design calculations:

Reactions at column bases:

$H_l = \dfrac{q_d * l^2}{4 * h * (2 * k + 3)}$ = 6,82 kN

$V_l = q_d * \dfrac{l}{2}$ = 36,00 kN

$M_{Bl} = -H_l * h$ = -40,92 kNm

$H_2 = H_{Ed} / 2$ = 4,68 kN

$V_2 = H_{Ed} * \dfrac{h}{l}$ = 7,02 kN

$M_{B2} = H_2 * h$ = 28,08 kNm

Analysis of column

$N_{Ed} = V_l + V_2$ = 43,02 kN

$V_{Ed} = H_l + H_2$ = 11,50 kN

Dischingerfaktor:

$D = \dfrac{1}{1 - \dfrac{d}{h * 100} * \dfrac{V}{H}}$ = 1,057

$M_D = M_{Bl} - D * M_{B2}$ = -70,60 kNm

Buckling in the plane of frame:

$k_c = \dfrac{I_{y2}}{h * 100}$ = 18,77 cm³

$k_1 = \dfrac{I_{y1}}{l * 100}$ = 4,86 cm³

$k_2 =$ 0,0

$h_2 =$ 1 (Hinge)

$h_1 = \dfrac{k_c}{k_c + k_1 + k_2}$ = 0,79

Slenderness ratio as in Fig. E.2.1.:

$l = 0,92 * h$ = 5,52 m

$l_y = 100 * \dfrac{l}{i_y}$ = 53,59

$l_{trans,y} = \dfrac{l_y}{l_1 * e}$ = 0,57

$\dfrac{h_2}{b_2}$ = 1,00 < 1,2

Apply strut curve b:

$a =$ 0,34

$b_A =$ 1,000

$j = 0,5*(1+ a *(l_{trans,y}-0,2)+ l_{trans,y}^2)$ = 0,725

$c_y = \dfrac{1}{j + Ö\overline{j^2 - l_{trans,y}^2}}$ = 0,8525

Buckling outside the plane of frame

$$l_z = 100 * \frac{h}{i_z} = 98{,}68$$

$$l_{trans,z} = \frac{l_z}{l_1 * e} = 1{,}05$$

Apply strut curve c:

$a =$ 0,49

$b_A =$ 1,000

$$j = 0{,}5 * (1 + a * (l_{trans,z} - 0{,}2) + l_{trans,z}^2) = 1{,}260$$

$$c_z = \frac{1}{j + \sqrt{j^2 - l_{trans,z}^2}} = 0{,}5111$$

$$c_{min} = \mathrm{MIN}(c_y;\ c_z) = 0{,}5111$$

Check bending and buckling:

$y =$ 0,00

$$b_{My} = 1{,}8 - 0{,}7 * y = 1{,}80$$

$$m_y = l_{trans,y} * (2 * b_{My} - 4) + \frac{W_{ply2} - W_{ely2}}{W_{ely2}} = 0{,}022 < 0{,}9$$

$$k_y = 1 - \frac{m_y * N_{Ed}}{c_y * A_2 * f_y} = 1{,}000 < 1{,}5$$

$$\frac{N_{Ed}}{c_{min} * A_2 * \frac{f_y}{g_M * 10}} + 100 * k_y * \frac{\mathrm{abs}(M_D)}{W_{ply2} * \frac{f_y}{g_M * 10}} = \underline{\mathbf{0{,}32 < 1}}$$

Check torsional-flexural buckling:

$C_1 =$ 1,879

$$M_{cr} = C_1 * \frac{p^2 * E_s * I_{z2}}{(h * 100)^2 * 10} * \sqrt{\frac{I_w}{I_{z2}} + \frac{(h * 100)^2 * G * I_{t2}}{p^2 * E_s * I_{z2}}} = 94241{,}31 \text{ kNm}$$

As in Table: F.1.1

$y =$ 0,00

$b_w =$ 1,00

$k =$ 1,00

$$l_{LT} = \sqrt{p^2 * \frac{E_s}{10} * \frac{W_{ply2}}{M_{cr}}} = 50{,}78$$

$$l_{trans,LT} = \frac{l_{LT}}{l_1} * \sqrt{b_w} = 0{,}541$$

Apply strut curve a

$a =$ 0,21

$$j = 0{,}5 * (1 + a * (l_{trans,LT} - 0{,}2) + l_{trans,LT}^2) = 0{,}682$$

$\chi_{LT} = \dfrac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{trans,LT}^2}}$ = 0,911

$\beta_{M,LT} = 1{,}8 - 0{,}7 * \psi$ = 1,80

$\mu_{LT} = 0{,}15 * \lambda_{trans,z} * \beta_{M,LT} - 0{,}15$ = 0,134 < 0,9

$k_{LT} = 1 - \dfrac{\mu_{LT} * N_{Ed}}{\chi_{LT} * A_2 * f_y}$ = 1,000 < 1,5

$\dfrac{N_{Ed}}{\chi_z * A_2 * \dfrac{f_y}{\gamma_M * 10}} + 100 * k_{LT} * \dfrac{abs(M_D)}{\chi_{LT} * W_{ply2} * \dfrac{f_y}{\gamma_M * 10}}$ = **<u>0,347 < 1</u>**

Section analysis at D:

Column:

$A_v = 1{,}04 * h_2 * s_2 / 10^2$ = 24,96 cm²

$V_{pl,Rd} = A_v * \dfrac{f_y}{\sqrt{3} * \gamma_M * 10}$ = 307,86 kN

$\dfrac{V_{Ed}}{V_{pl,Rd}}$ = **<u>0,037 < 0,5</u>**

No interaction

$N_{pl,Rd} = A_2 * \dfrac{f_y}{\gamma_M * 10}$ = 2264,55 kN

$n = \dfrac{N_{Ed}}{N_{pl,Rd}}$ = 0,02

$M_{pl,y,Rd} = W_{ply2} * \dfrac{f_y}{\gamma_M * 10^3}$ = 250,49 kNm

$M_{Ny,Rd} = 1{,}11 * M_{pl,y,Rd} * (1 - n)$ = 272,48 kNm

$MAX\left(\dfrac{abs(M_D)}{M_{pl,y,Rd}} ; \dfrac{abs(M_D)}{M_{Ny,Rd}}\right)$ = **<u>0,28 < 1</u>**

Beam:

$A_v = 1{,}04 * h_1 * s_1 / 10^2 = 15{,}48\ cm^2$

$V_{pl,Rd} = A_v * \frac{f_y}{\sqrt{3} * \gamma_M * 10} = 190{,}93\ kN$

$\frac{N_{Ed}}{V_{pl,Rd}} =$ **0,225 < 0,5**

No interaction

$N_{pl,Rd} = A_1 * f_y / 10 / \gamma_M = 835{,}32\ kN$

$n = V_{Ed} / N_{pl,Rd} = 0{,}014$

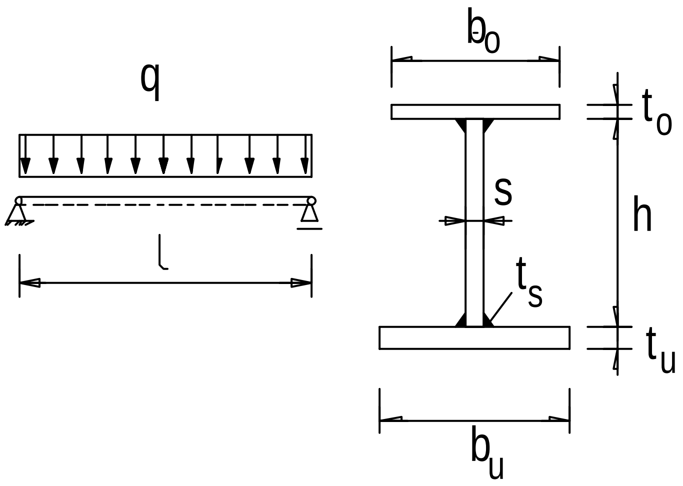
$M_{pl,y,Rd} = W_{ply1} * \frac{f_y}{\gamma_M * 10^3} = 78{,}91\ kNm$

$M_{Ny,Rd} = 1{,}11 * M_{pl,y,Rd} * (1 - n) = 86{,}36\ kNm$

$MAX\left(\frac{abs(M_D)}{M_{pl,y,Rd}} ; \frac{abs(M_D)}{M_{Ny,Rd}}\right) =$ **0,89 < 1**

Pos.: Flexural-torsional buckling of a single-span beam:

End supports are laterally fixed.



Load diagram:

Span length l =	24,00 m
top width b_o=	40,00 cm
bottom width b_u=	50,00 cm
Depth of web h=	100,00 cm
top Flange thickness t_o=	3,50 cm
bottom Flange thickness t_u=	3,50 cm
Web thickness s=	2,00 cm
Weld thickness t_s=	1,00 cm

Loads:

q = 4,04 kN/m

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 510
E_s=	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
G=	TAB("steel/EC"; G; Name=steel)	=	81000,00 N/mm²
f_y=	TAB("steel/EC"; fy; Name=steel)	=	355,00 N/mm²
ε =	$\sqrt{\dfrac{235}{f_y}}$	=	0,81

Partial safety factors:

γ_M= 1,10

γ_g= 1,35

Section classification as in Table 5.3.1:

A =	$(b_o * t_o + b_u * t_u + h * s)$	=	515,00 cm²

Plastic neutral axis:

x =	$\dfrac{\dfrac{A}{2} - b_o * t_o}{s}$	=	58,75 cm
x_s =	h - x	=	41,25 cm

Flange:

$$c = \frac{\frac{b_o - s}{2} - t_s * \sqrt{2}}{t_o} = 5{,}02 \text{ cm}$$

$$\frac{c}{9 * \varepsilon} = 0{,}69 < 1$$

$\Rightarrow$ Section class 1

Web:

$$\alpha = \frac{x}{h} = 0{,}59 \text{ cm}$$

$$\frac{h}{s * 456 * \frac{\varepsilon}{13 * \alpha - 1}} = 0{,}90 < 1$$

$$\frac{h}{s * 396 * \frac{\varepsilon}{13 * \alpha - 1}} = 1{,}04 > 1$$

$\Rightarrow$ Section class 2

<u>**Check bending moment strength:**</u>

$$M_{Ed} = \gamma_g * \frac{q * l^2}{8} = 392{,}69 \text{ kNm}$$

Critical torsional-buckling moment:

$$I_z = t_o * \frac{b_o^3}{12} + t_u * \frac{b_u^3}{12} = 55125{,}00 \text{ cm}^4$$

As in Table: F.1.2(2)

$k =$ 1,00

As in Table: F.1.1(4)

$k_w =$ 1,00

As in Table: F.1.2

$C_1 =$ 1,132

$C_2 =$ 0,459

$C_3 =$ 0,525

Location of center of gravity:

$$e = \frac{b_u * t_u * (h + t_o) + h * s * \frac{(h + t_o)}{2}}{A} = 55{,}27 \text{ cm}$$

Location of shear centre:

$s_s = \left(h + \frac{t_o + t_u}{2}\right) * \frac{b_u^{\,3}}{b_o^{\,3} + b_u^{\,3}}$ = 68,45 cm

$z_s = -(s_s - e)$ = -13,18 cm

$I_t = \frac{1}{3} * (b_u * t_u^3 + b_o * t_o^3 + h * s^3)$ = 1552,92 cm⁴

$z_a =$ 0,00 cm

$z_g = z_a - z_s$ = 13,18 cm

as in F 1.4:

$b_f = \frac{b_o^{\,3}}{b_o^{\,3} + b_u^{\,3}}$ = 0,339 < 0,5

$z_j = (2 * b_f - 1) * \frac{h + \frac{t_o + t_u}{2}}{2}$ = -16,66 cm

$I_w = b_f * (1 - b_f) * I_z * \left(h + \frac{t_o + t_u}{2}\right)^2$ = 132,32*10⁶ cm⁶

$P_1 = C_1 * \frac{p^2 * E_s * I_z}{(k * l * 100)^2}$ = 22453,85

$P_2 = \sqrt{\left(\frac{k}{k_w}\right)^2 * \frac{I_w}{I_z} + \frac{(k * l * 100)^2 * G * I_t}{p^2 * E_s * I_z} + (C_2 * z_g - C_3 * z_j)^2}$ = 94,66

$P_3 = C_2 * z_g - C_3 * z_j$ = 14,80

$M_{cr} = P_1 * \frac{P_2 - P_3}{10^3}$ = 1793,16 kNm

$W_{pl} = b_u * t_u * (x_s + \frac{t_u}{2}) + x_s^2 * \frac{s}{2} + b_o * t_o * (x + \frac{t_o}{2}) + x^2 * \frac{s}{2}$ = 21148,13 cm³

Section class 2 $b_w =$ 1,00

$l_{trans,LT} = \sqrt{b_w * W_{pl} * \frac{f_y}{10^3 * M_{cr}}}$ = 2,046

As in Table 5.5.1 $a =$ 0,49

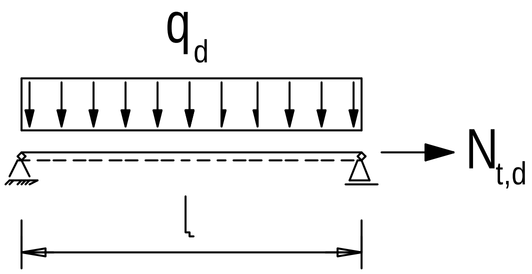
$j = 0,5 * (1 + a * (l_{trans,LT} - 0,2) + l_{trans,LT}^2)$ = 3,05

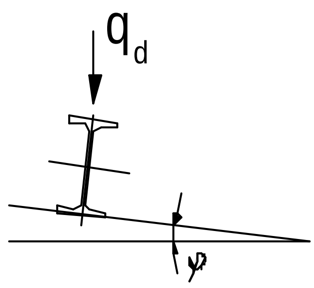
$c_{LT} = \frac{1}{j + \sqrt{j^2 - l_{trans,LT}^{\,2}}}$ = 0,1883

$M_{b,Rd} = b_w * c_{LT} * W_{pl} * \frac{f_y}{g_M^{\,2} * 10^3}$ = 1168,33 kNm

$\frac{M_{Ed}}{M_{b,Rd}}$ = **0,34 < 1**

Pos.: Single-span beam with biaxial bending:





Span length l =	5,80 m
Angle of slope j =	8,53 °

Loads:

q_d=	7,00 kN/m
$N_{t,d}$=	100,00 kN

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s=	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
f_y=	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
ε =	$\ddot{O}\overline{\frac{235}{f_y}}$	=	1,00

platischer Formbeiwert a_{ply} =	1,14
platischer Formbeiwert a_{plz} =	1,25
g_M=	1,10

Profil Typ =	SEL("steel/Profils"; Name;)	=	HEA
Selected Profil =	SEL("steel/"Typ; Name;)	=	HEA 140

Section classification:

Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	133,00 mm
Depth of web h_1 =	TAB("steel/"Typ; h1; Name=Profil)	=	92,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	5,50 mm
Flange width b =	TAB("steel/"Typ; b; Name=Profil)	=	140,00 mm
Flange thickness t =	TAB("steel/"Typ; t; Name=Profil)	=	8,50 mm
Moment of inertia I =	TAB("steel/"Typ; Iy; Name=Profil)	=	1030,00 cm⁴
Cross-sectional area A=	TAB("steel/"Typ; A; Name=Profil)	=	31,40 cm²
W_{ely}=	TAB("steel/"Typ; Wy; Name=Profil)	=	155,00 cm³
W_{ply} =	$a_{ply} * W_{ely}$	=	176,70 cm³
W_{elz}=	TAB("steel/"Typ; Wz; Name=Profil)	=	55,60 cm³
W_{plz} =	$a_{plz} * W_{elz}$	=	69,50 cm³

Design loads:

Plastic limit force:

$N_{pl,Rd} = A * \frac{f_y}{\gamma_M * 10}$ = 670,82 kN

Limit force that plasticizes the web

$N_{pl,w,Rd} = (h - 2 * t) * s * \frac{f_y}{\gamma_M * 10^3}$ = 136,30 kN

$MAX\left(\frac{N_{t,d}}{0,5 * N_{pl,w,Rd}} ; \frac{N_{t,d}}{0,25 * N_{pl,Rd}}\right)$ = **1,47 > 1**

$\Rightarrow$ Reduce plastic limit force as in 5.4.8.1:

$\alpha = \frac{A - 2 * b * \frac{t}{100}}{A}$ = 0,242 < 0,5

$\eta = \frac{N_{t,d}}{N_{pl,Rd}}$ = 0,149

$M_{Ny,Rd} = W_{ply} * \frac{f_y}{\gamma_M * 10^3} * \frac{1 - \eta}{1 - 0,5 * \alpha}$ = 36,55 kNm

$\frac{\eta}{\alpha}$ = 0,62 < 1

$\Rightarrow$ $M_{Nz,Rd} = W_{plz} * \frac{f_y}{\gamma_M * 10^3}$ = 14,85 kNm

Design momente:

$q_{dy} = q_d * COS(\varphi)$ = 6,92 kN/m

$q_{dz} = q_d * SIN(\varphi)$ = 1,04 kN/m

$M_{y,Ed} = q_{dy} * \frac{l^2}{8}$ = 29,10 kNm

$M_{z,Ed} = q_{dz} * \frac{l^2}{8}$ = 4,37 kNm

Check section strength:

for I-Profile gilt $\alpha =$ 2,00

$\beta_1 = 5 * \eta$ = 0,75 < 1

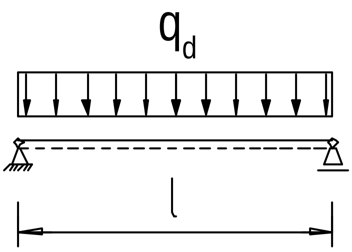
$\beta_2 =$ 1,00

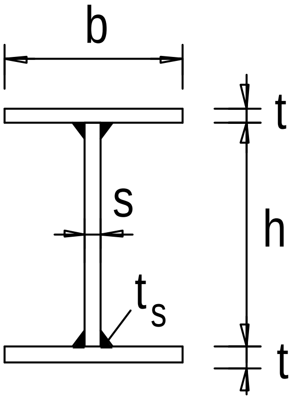
$\beta = MAX(\beta_1;\beta_2)$ = 1,00

$\left(\frac{M_{y,Ed}}{M_{Ny,Rd}}\right)^{\alpha} + \left(\frac{M_{z,Ed}}{M_{Nz,Rd}}\right)^{\beta}$ = **0,93 < 1**

Pos.: flexural-torsional buckling:

End supports are laterally fixed, load is applied in the middle of the bottom flange.





Load diagram:

Span length l =	8,00 m
Beam width b=	20,00 cm
Column height h=	50,00 cm
Flange thickness t=	2,50 cm
Web thickness s=	1,20 cm
Weld thickness t_s=	0,50 cm

Loads:

q_d =	50,00 kN/m

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 510
E_s=	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
G=	TAB("steel/EC"; G; Name=steel)	=	81000,00 N/mm²
f_y=	TAB("steel/EC"; fy; Name=steel)	=	355,00 N/mm²

$$\varepsilon = \sqrt{\frac{235}{f_y}} = 0{,}81$$

Partial safety factors:

γ_M= 1,10

Section classification:

Flange:

$$c = \frac{\frac{b}{2} - t_s * \sqrt{2}}{t} = 3{,}72 \text{ cm}$$

$$\frac{c}{9 * \varepsilon} = 0{,}51 < 1$$

$\Rightarrow$ Section class 1

Web:

$$d = h - 2 * t_s * \sqrt{2} = 48{,}59 \text{ cm}$$

$$\frac{d}{s * 72 * \varepsilon} = 0{,}69 < 1$$

$\Rightarrow$ Section class 1

Check bending moment strength:

$$W_{pl} = \left(b * t * \frac{h+t}{2} + \frac{h}{2} * s * \frac{h}{4}\right) * 2 \quad = \quad 3375,00 \text{ cm}^3$$

$$M_{c,Rd} = \frac{W_{pl} * f_y}{\gamma_M * 10^3} \quad = \quad 1089,20 \text{ kNm}$$

$$M_{Ed} = \frac{q_d * l^2}{8} \quad = \quad 400,00 \text{ kNm}$$

$$\frac{M_{Ed}}{M_{c,Rd}} = \underline{0,37 < 1}$$

Check shear strength:

$$A_v = h * s \quad = \quad 60,00 \text{ cm}^2$$

$$V_{pl,Rd} = A_v * \frac{f_y}{\sqrt{3} * \gamma_M * 10} \quad = \quad 1117,96 \text{ kN}$$

$$V_{Ed} = q_d * \frac{l}{2} \quad = \quad 200,00 \text{ kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \underline{0,18 < 1}$$

Check torsional buckling strength:

$$I_z = 2 * t * \frac{b^3}{12} \quad = \quad 3333,33 \text{ cm}^4$$

$$I_w = I_z * \frac{(h+t)^2}{4} \quad = \quad 2,297*10^6 \text{ cm}^6$$

$$I_t = \frac{1}{3} * (2 * b * t^3 + h * s^3) \quad = \quad 237,13 \text{ cm}^4$$

As in Table F.1.2(2) k=	1,00
As in Table F.1.1(4) k_W=	1,00
As in Table F.1.2 C_1=	1,13
As in Table F.1.2 C_2=	0,46
As in Table F.1.2 C_3=	0,53
Load is applied in centroid of section z_a=	-27,50 cm
Centroid of section and shear centre are identical z_S=	0,00 cm
Section is symmetrical z_j=	0,00 cm
z_g= z_a-z_S =	-27,50 cm

Critical torsional-buckling moment

$$P_1 = C_1 * \frac{\pi^2 * E_s * I_z}{(k * l * 100)^2} = 12198{,}18$$

$$P_2 = \sqrt{\left(\frac{k}{k_w}\right)^2 * \frac{I_w}{I_z} + \frac{(k * l * 100)^2 * G * I_t}{\pi^2 * E_s * I_z} + (C_2 * z_g - C_3 * z_j)^2} = 51{,}27$$

$$P_3 = C_2 * z_g - C_3 * z_j = -12{,}65$$

$$M_{cr} = P_1 * \frac{P_2 - P_3}{10^3} = 779{,}71 \text{ kNm}$$

Cross-section class 1 $\beta_w = 1{,}00$

$$\lambda_{trans,LT} = \sqrt{\beta_w * W_{pl} * \frac{f_y}{10^3 * M_{cr}}} = 1{,}240$$

Apply strut curve c:

$\alpha_{LT} =$ 0,49

$$\phi_{LT} = 0{,}5*(1+\alpha_{LT}*(\lambda_{trans,LT}-0{,}2)+\lambda_{trans,LT}^2) = 1{,}524$$

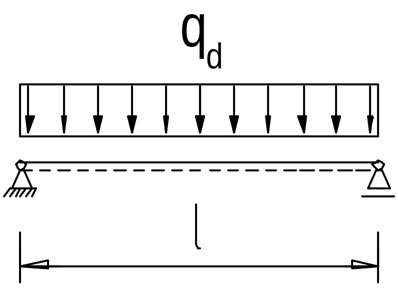
$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{trans,LT}^2}} = 0{,}415$$

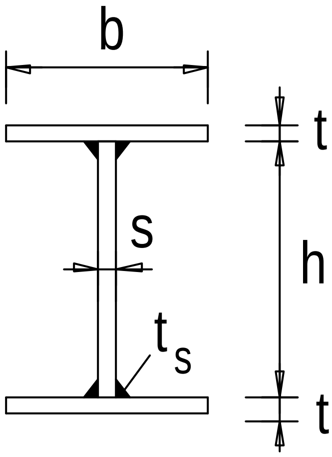
$$M_{b,Rd} = \beta_w * \chi_{LT} * W_{pl} * \frac{f_y}{\gamma_M * 10^3} = 452{,}02 \text{ kNm}$$

$$\frac{M_{Ed}}{M_{b,Rd}} = \underline{\mathbf{0{,}88 < 1}}$$

Pos.: Flexural-torsional buckling of a single-span beam:

End supports are laterally fixed, Load is applied in centroid of section.





Load diagram:

Span length l =	8,00 m
Beam width b=	20,00 cm
Column height h=	50,00 cm
Flange thickness t=	2,50 cm
Web thickness s=	1,20 cm
Weld thickness t_s=	0,50 cm

Loads:

q_d =	45,00 kN/m

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 510
E_s=	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
G=	TAB("steel/EC"; G; Name=steel)	=	81000,00 N/mm²
f_y=	TAB("steel/EC"; fy; Name=steel)	=	355,00 N/mm²

$$\varepsilon = \sqrt{\frac{235}{f_y}} = 0,81$$

Partial safety factors:

γ_M= 1,10

Section classification As in Table 5.3.1:

Flange:

$$c = \frac{\frac{b}{2} - t_s * \sqrt{2}}{t} \quad = \quad 3{,}72 \text{ cm}$$

$$\frac{c}{9 * \varepsilon} \quad = \quad 0{,}51 < 1$$

Section class 1

Web:

$$d = h - 2 * t_s * \sqrt{2} \quad = \quad 48{,}59 \text{ cm}$$

$$\frac{d}{s * 72 * \varepsilon} \quad = \quad 0{,}69 < 1$$

Section class 1

Check bending moment strength:

$$W_{pl} = \left(b * t * \frac{h + t}{2} + \frac{h}{2} * s * \frac{h}{4} \right) * 2 \quad = \quad 3375{,}00 \text{ cm}^3$$

$$M_{c,Rd} = \frac{W_{pl} * f_y}{\gamma_M * 10^3} \quad = \quad 1089{,}20 \text{ kNm}$$

$$M_{Ed} = \frac{q_d * l^2}{8} \quad = \quad 360{,}00 \text{ kNm}$$

$$\frac{M_{Ed}}{M_{c,Rd}} \quad = \quad \underline{0{,}33 < 1}$$

Check shear strength:

$$A_v = h * s \quad = \quad 60{,}00 \text{ cm}^2$$

$$V_{pl,Rd} = A_v * \frac{f_y}{\sqrt{3} * \gamma_M * 10} \quad = \quad 1117{,}96 \text{ kN}$$

$$V_{Ed} = q_d * \frac{l}{2} \quad = \quad 180{,}00 \text{ kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} \quad = \quad \underline{0{,}16 < 1}$$

Check torsional buckling strength:

$$I_z = 2 * t * \frac{b^3}{12} = 3333{,}33\ cm^4$$

$$I_w = I_z * \frac{(h+t)^2}{4} = 2{,}297 * 10^6\ cm^6$$

$$I_t = \frac{1}{3} * (2 * b * t^3 + h * s^3) = 237{,}13\ cm^4$$

As in Table F.1.2(2) k= 1,00

As in Table F.1.1(4) k_W= 1,00

As in Table F.1.2 C_1= 1,13

As in Table F.1.2 C_2= 0,46

As in Table F.1.2 C_3= 0,53

Load in centroid z_a= 0,00 cm

Centroid of section and shear centre are identical z_S= 0,00 cm

Section is symmetrical z_j= 0,00 cm

Critical torsional-buckling moment

$$M_{cr} = C_1 * \frac{\pi^2 * E_s * I_z}{(k * l * 10^2)^2} * \frac{\sqrt{\left(\frac{k}{k_w}\right)^2 * \frac{I_w}{I_z} + \frac{(k * l * 10^2)^2 * G * I_t}{\pi^2 * E_s * I_z}}}{10^3} = 606{,}05\ kNm$$

Cross-section class 1 β_w = 1,00

$$\bar{\lambda}_{trans,LT} = \sqrt{\beta_w * W_{pl} * \frac{f_y}{M_{cr} * 10^3}} = 1{,}406$$

Apply strut curve c:

α_{LT}= 0,49

$$\phi_{LT} = 0{,}5*(1+\alpha_{LT}*(\bar{\lambda}_{trans,LT}-0{,}2)+\bar{\lambda}_{trans,LT}^2) = 1{,}784$$

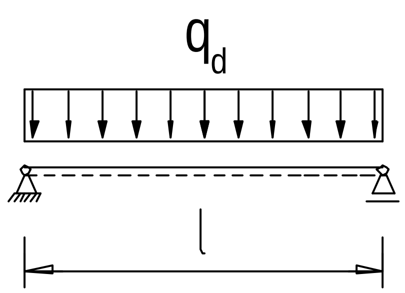
$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{trans,LT}^2}} = 0{,}347$$

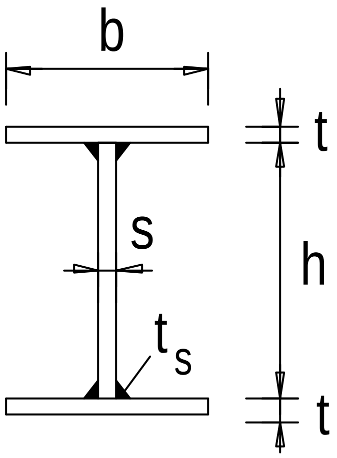
$$M_{b,Rd} = \beta_w * \chi_{LT} * W_{pl} * \frac{f_y}{\gamma_M * 10^3} = 377{,}95\ kNm$$

$$\frac{M_{Ed}}{M_{b,Rd}} = \underline{\mathbf{0{,}95 < 1}}$$

Pos.: Flexural-torsional buckling of a single-span beam:

End supports are laterally fixed, Load is applied in centroid of section.





Load diagram:

Span length l =	8,00 m
Beam width b=	20,00 cm
Column height h=	50,00 cm
Flange thickness t=	2,50 cm
Web thickness s=	1,20 cm
Weld thickness t_s=	0,50 cm

Loads:

q_d =	36,00 kN/m

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 510
E_s=	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
G=	TAB("steel/EC"; G; Name=steel)	=	81000,00 N/mm²
f_y=	TAB("steel/EC"; fy; Name=steel)	=	355,00 N/mm²
$\varepsilon =$	$\sqrt{\frac{235}{f_y}}$	=	0,81

Partial safety factors:

γ_M= 1,10

Section classification As in Table 5.3.1:

Flange:

$$c = \frac{\frac{b}{2} - t_s \cdot \sqrt{2}}{t} = 3{,}72 \text{ cm}$$

$$\frac{c}{9 \cdot \varepsilon} = 0{,}51 < 1$$

Section class 1

Web:

$$d = h - 2 \cdot t_s \cdot \sqrt{2} = 48{,}59 \text{ cm}$$

$$\frac{d}{s \cdot 72 \cdot \varepsilon} = 0{,}69 < 1$$

Section class 1

Check bending moment strength:

$$W_{pl} = \left(b \cdot t \cdot \frac{h+t}{2} + \frac{h}{2} \cdot s \cdot \frac{h}{4} \right) \cdot 2 = 3375{,}00 \text{ cm}^3$$

$$M_{c,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_M \cdot 10^3} = 1089{,}20 \text{ kNm}$$

$$M_{Ed} = \frac{q_d \cdot l^2}{8} = 288{,}00 \text{ kNm}$$

$$\frac{M_{Ed}}{M_{c,Rd}} = \underline{\mathbf{0{,}26 < 1}}$$

Check shear strength:

$$A_v = h \cdot s = 60{,}00 \text{ cm}^2$$

$$V_{pl,Rd} = A_v \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_M \cdot 10} = 1117{,}96 \text{ kN}$$

$$V_{Ed} = q_d \cdot \frac{l}{2} = 144{,}00 \text{ kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \underline{\mathbf{0{,}13 < 1}}$$

Check torsional buckling strength:

$I_z = 2 * t * \frac{b^3}{12}$ = 3333,33 cm^4

$I_w = I_z * \frac{(h+t)^2}{4}$ = $2{,}297*10^6$ cm^6

$I_t = \frac{1}{3} * (2 * b * t^3 + h * s^3)$ = 237,13 cm^4

As in Table F.1.2(2) k= 1,00

As in Table F.1.1(4) k_w= 1,00

As in Table F.1.2 C_1= 1,13

As in Table F.1.2 C_2= 0,46

As in Table F.1.2 C_3= 0,53

Load in centroid z_a= 27,50 cm

Centroid of section and shear centre are identical z_s= 0,00 cm

Section is symmetrical z_j= 0,00 cm

$z_g = z_a - z_s$ = 27,50 cm

Critical torsional-buckling moment

$P_1 = C_1 * \frac{\pi^2 * E_s * I_z}{(k * l * 100)^2}$ = 12198,18

$P_2 = \sqrt{\left(\frac{k}{k_w}\right)^2 * \frac{I_w}{I_z} + \frac{(k * l * 100)^2 * G * I_t}{\pi^2 * E_s * I_z} + (C_2 * z_g - C_3 * z_j)^2}$ = 51,27

$P_3 = C_2 * z_g - C_3 * z_j$ = 12,65

$M_{cr} = P_1 * \frac{P_2 - P_3}{10^3}$ = 471,09 kNm

Cross-section class 1 β_w= 1,00

$\lambda_{trans,LT} = \sqrt{\beta_w * W_{pl} * \frac{f_y}{10^3 * M_{cr}}}$ = 1,595

Apply strut curve c:

α_{LT}= 0,49

$\phi_{LT} = 0{,}5*(1+\alpha_{LT}*(\lambda_{trans,LT}-0{,}2)+\lambda_{trans,LT}^2)$ = 2,114

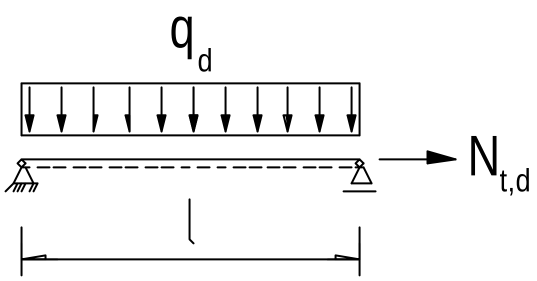
$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{trans,LT}^2}}$ = 0,286

$M_{b,Rd} = \beta_w * \chi_{LT} * W_{pl} * \frac{f_y}{\gamma_M * 10^3}$ = 311,51 kNm

$\frac{M_{Ed}}{M_{b,Rd}}$ = **0,92 < 1**

Pos.: Single-span beam subject to flexural-torsional buckling with uniform load and compression:

End supports are laterally fixed.



Load diagram:

Span length l =	24,00 m
top Flange width b_o=	40,00 cm
bottom Flange width b_u=	50,00 cm
Depth of web h=	100,00 cm
top Flange thickness t_o=	3,50 cm
bottom Flange thickness t_u=	3,50 cm
Web thickness s=	2,00 cm
Weld thickness t_s=	1,00 cm

Loads:

q_d =	18,00 kN/m
$N_{t,d}$=	500,00 kN

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 510
E_s=	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
G =	TAB("steel/EC"; G; Name=steel)	=	81000,00 N/mm²
f_y=	TAB("steel/EC"; fy; Name=steel)	=	355,00 N/mm²
ε =	$\sqrt{\frac{235}{f_y}}$	=	0,81

Partial safety factors:

g_M=	1,10
g_g=	1,35

A = $(b_o * t_o + b_u * t_u + h * s)$ = 515,00 cm²

Location of center of gravity:

$$e = \frac{b_u * t_u * (h + t_o) + h * s * \frac{(h + t_o)}{2}}{A} = 55,27 \text{ cm}$$

Location of shear centre:

$$s_s = \left(h + \frac{t_o + t_u}{2}\right) * \frac{b_u^{\,3}}{b_o^{\,3} + b_u^{\,3}} = 68,45 \text{ cm}$$

Plastic neutral axis:

$$x = \frac{\frac{A}{2} - (b_o * t_o)}{s} = 58{,}75 \text{ cm}$$

<u>Section classification As in Table 5.3.1</u>:

Flange:

$$c = \frac{\frac{b_o - s}{2} - t_s * \sqrt{2}}{t_o} = 5{,}02 \text{ cm}$$

$$\frac{c}{9 * \varepsilon} = 0{,}69 < 1$$

Section class 1

Web:

$$\alpha = \frac{x}{h} = 0{,}59 \text{ cm}$$

$$\frac{h}{s * 456 * \frac{\varepsilon}{13 * \alpha - 1}} = 0{,}90 < 1$$

$$\frac{h}{s * 396 * \frac{\varepsilon}{13 * \alpha - 1}} = 1{,}04 > 1$$

Section class 2

<u>Check bending moment strength:</u>

$$M_{Ed} = \frac{q_d * l^2}{8} = 1296{,}00 \text{ kNm}$$

$$I_z = t_o * \frac{b_o^3}{12} + t_u * \frac{b_u^3}{12} = 55125{,}00 \text{ cm}^4$$

$$I_y = b_o*t_o*e^2+b_u*t_u*(h+t_u-e)^2+s*h^3/12+h*s*(e-(h/2+t_u/2))^2 = 1003886{,}210 \text{ cm}^4$$

$$I_y = b_o*t_o*e^2+b_u*t_u*(h+t_u-e)^2+s*\frac{h^3}{12}+h*s*\left(e - \left(\frac{h}{2}+\frac{t_u}{2}\right)\right)^2 = 1003886{,}210 \text{ cm}^4$$

$$W_{com} = I_y / (e + t_o / 2) = 17605{,}86 \text{ cm}^3$$

Effective bending moment:

$M_{Ed} = \dfrac{q_d * l^2}{8}$ = 1296,00 kNm

Assumption as in 5.5.3 ψ_{vec} = 0,80

$\sigma_{com,Ed} = \dfrac{M_{Ed} * 100}{W_{com}} - \psi_{vec} * \dfrac{N_{t,d}}{A}$ = 6,58 kN/cm²

$M_{eff,Ed} = \dfrac{W_{com} * \sigma_{com,Ed}}{100}$ = 1158,47 kNm

As in TableF.1.2(2) k = 1,00

As in TableF.1.1(4) k_w = 1,00

As in Table: F.1.2

C_1 = 1,132

C_2 = 0,459

C_3 = 0,525

$z_s = -(s_s - e)$ = -13,18 cm

$I_t = \dfrac{1}{3} * (b_u * t_u^3 + b_o * t_o^3 + h * s^3)$ = 1552,92 cm⁴

z_a = 0,00 cm

$z_g = z_a - z_s$ = 13,18 cm

$\beta_f = \dfrac{b_o^3}{b_o^3 + b_u^3}$ = 0,339 < 0,5

$z_j = (2 * \beta_f - 1) * \dfrac{h + \dfrac{t_o + t_u}{2}}{2}$ = -16,66 cm

$I_w = \beta_f * (1 - \beta_f) * I_z * \left(h + \dfrac{t_o + t_u}{2}\right)^2$ = 132,32*10⁶ cm⁶

$P_1 = C_1 * \dfrac{\pi^2 * E_s * I_z}{(k * l * 100)^2}$ = 22453,85

$P_2 = \sqrt{\left(\dfrac{k}{k_w}\right)^2 * \dfrac{I_w}{I_z} + \dfrac{(k * l * 100)^2 * G * I_t}{\pi^2 * E_s * I_z} + (C_2 * z_g - C_3 * z_j)^2}$ = 94,66

$P_3 = C_2 * z_g - C_3 * z_j$ = 14,80

$M_{cr} = P_1 * \dfrac{P_2 - P_3}{10^3}$ = 1793,16 kNm

$W_{pl} = b_u * t_u * (x + \dfrac{t_u}{2}) + x^2 * \dfrac{s}{2} + b_o * t_o * (h\text{-}x + \dfrac{t_o}{2}) + (h\text{-}x)^2 * \dfrac{s}{2}$ = 21760,63 cm³

Section class 2 β_w = 1,00

$\lambda_{trans,LT} = \sqrt{\beta_w * W_{pl} * \dfrac{f_y}{10^3 * M_{cr}}}$ = 2,076

As in Table 5.5.1 $\alpha =$ 0,49

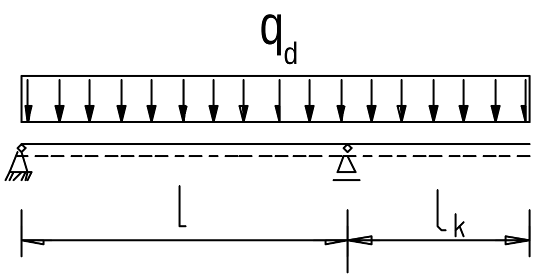
$\phi = 0{,}5 \cdot (1 + \alpha \cdot (\bar{\lambda}_{trans,LT} - 0{,}2) + \bar{\lambda}_{trans,LT}^2) = 3{,}11$

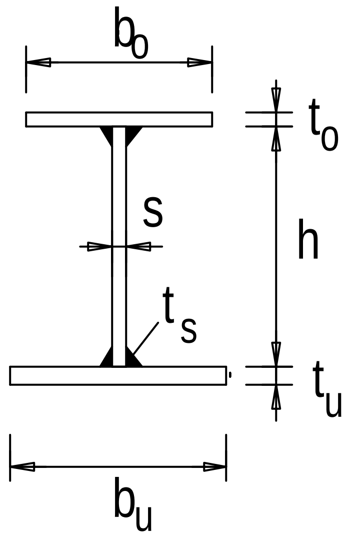
$$\chi_{LT} = \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans,LT}^2}} = 0{,}1843$$

$$M_{b,Rd} = \beta_w \cdot \chi_{LT} \cdot W_{pl} \cdot \frac{f_y}{\gamma_M^2 \cdot 10^3} = 1176{,}63 \text{ kNm}$$

$$\frac{M_{eff,Ed}}{M_{b,Rd}} = \underline{\mathbf{0{,}98 < 1}}$$

Pos.: Single-span beam with cantilever, asymmetric section:





Span length l =	10,50 m
Cantilever length l_k=	5,00 m
top Flange width b_o =	50,00 cm
bottom Flange width b_u =	30,00 cm
top Flange thickness t_o =	2,00 cm
bottom Flange thickness $_{tu}$ =	2,00 cm
Depth of web h =	60,00 cm
Web thickness s =	1,50 cm
Weld thickness t_s =	0,80 cm

Loads:

Dead weight g =	26,00 kN/m
Live load q =	37,70 kN/m
Snow load s_0 =	7,00 kN/m

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s=	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
f_y=	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
$\varepsilon =$	$\sqrt{\frac{235}{f_y}}$	=	1,00

Partial safety factors:

g_{fg}=	1,35
g_{fq}=	1,50
y =	0,90
g_M=	1,10

Design loads:

q_{d1}=	g_{fg} * g + g_{fq} * q	=	91,65 kN/m
q_{d2}=	g_{fg} * g + y * g_{fq} * (q + s_0)	=	95,44 kN/m
q_d=	MAX(q_{d1}; q_{d2})	=	95,44 kN/m
M_{Edk}=	$-q_d * \frac{l_k^2}{2}$	=	-1193,00 kNm
B_v=	$\frac{(l+l_k)^2}{2} * \frac{q_d}{l}$	=	1091,88 kN
B_r=	$q_d * l_k$	=	477,20 kN
B_l=	$B_v - B_r$	=	614,68 kN
A_v=	$(l + l_k) * q_d - B_v$	=	387,44 kN
$M_{Edfield}$=	$A_v * \frac{A_v}{q_d * 2}$	=	786,41 kNm

Plastic neutral axis:

A =	$(b_o * t_o + b_u * t_u + h * s)$	=	250,00 cm²
x =	$\frac{\frac{A}{2} - b_o * t_o}{s}$	=	16,67 cm
W_{pl} =	$(b_o*t_o*(x+\frac{t_o}{2})+0,5*s*x^2+b_u*t_u*(\frac{t_u}{2}+(h-x))+0,5*s*(h-x)^2)$	=	6043,33 cm³
M_{pl} =	$W_{pl} * \frac{f_y}{10^3}$	=	1420,18 kNm

Section classification Kragarm As in Table 5.3.1:

Bottom Flange:

c =	$\frac{b_u - s}{2} - t_s * Ö\overline{2}$	=	13,12 cm
$\frac{c}{9 * t_u * e}$		=	0,73 < 1

Web:

a =	$\frac{h - x}{h}$	=	0,72
$\frac{\left(\frac{h - 2 * t_s * Ö\overline{2}}{s}\right)}{\left(396 * \frac{e}{13 * a - 1}\right)}$		=	0,81 < 1

Þ Section class 1.

Check shear strength:

$A_V = \quad h * s \quad = \quad 90{,}00\ cm^2$

$V_{pl,Rd} = \dfrac{A_V * f_y}{10 * \sqrt{3} * \gamma_M} \quad = \quad 1110{,}09\ kN$

$V_{Ed} = \quad MAX(B_l;\ B_r;A_v) \quad = \quad 614{,}68\ kN$

$\dfrac{V_{Ed}}{V_{pl,Rd} * 0{,}5} \quad = \quad \underline{\mathbf{1{,}11 > 1}}$

$\Rightarrow$ Mpl, reduced.

Check bending moment strength:

$\rho = \left(2 * \dfrac{V_{Ed}}{V_{pl,Rd}} - 1\right)^2 \quad = \quad 0{,}012$

$M_{V,Rd} = \quad (1 - \rho) * \dfrac{M_{pl}}{\gamma_M} \quad = \quad 1275{,}58\ kNm$

$abs\left(\dfrac{M_{Edk}}{M_{V,Rd}}\right) \quad = \quad \underline{\mathbf{0{,}94 < 1}}$

Section classification Feld As in Table 5.3.1:

top Flange:

$c = \quad 0{,}5 * (b_o - s) - t_s * \sqrt{2} \quad = \quad 23{,}12\ cm$

$c\ /\ t_o\ /\ (10 * \varepsilon) \quad = \quad 1{,}16 > 1$

$\dfrac{c}{t_o * 10 * \varepsilon} \quad = \quad 1{,}16 > 1$

$\dfrac{c}{t_o * 14 * \varepsilon} \quad = \quad 0{,}83 < 1$

$\Rightarrow$ Section class 3.

Web:

$\alpha = \dfrac{x}{h} \quad = \quad 0{,}28 < 1$

$\dfrac{\left(\dfrac{h - 2 * t_s * \sqrt{2}}{s}\right)}{\left(396 * \dfrac{\varepsilon}{13 * \alpha - 1}\right)} \quad = \quad 0{,}26 < 1$

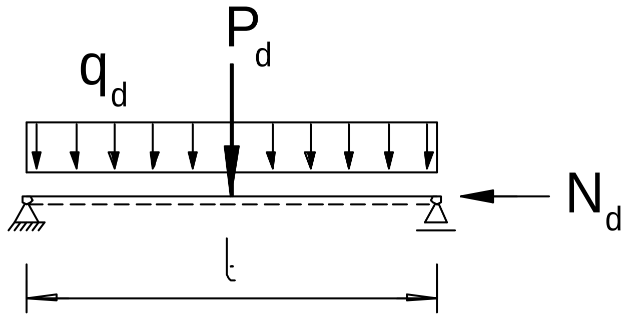
$\Rightarrow$ Section class 1.

Check section strength:

$\dfrac{abs\,(M_{Edfield})}{\dfrac{M_{pl}}{\gamma_M}} \quad = \quad \underline{\mathbf{0{,}61 < 1}}$

Pos.: Analysis single-span beam:

End supports are laterally fixed.



Load diagram:

Span length l = 6,00 m

Loads:

q_d =	10,00 kN/m
N_d=	273,00 kN
P_d=	15,00 kN

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s=	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
f_y=	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
ε =	$\sqrt{\frac{235}{f_y}}$	=	1,00

Partial safety factors:

g_M=	1,10
Plastic shape factor a_{pll} =	1,14

Section classification:

Profil Typ =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil =	SEL("steel/"Typ; Name;)	=	IPE 300
Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	300,00 mm
Depth of web h_1 =	TAB("steel/"Typ; h1; Name=Profil)	=	248,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	7,10 mm
Flange width b =	TAB("steel/"Typ; b; Name=Profil)	=	150,00 mm
Flange thickness t =	TAB("steel/"Typ; t; Name=Profil)	=	10,70 mm
Moment of inertia I =	TAB("steel/"Typ; Iy; Name=Profil)	=	8360,00 cm⁴
Cross-sectional area A=	TAB("steel/"Typ; A; Name=Profil)	=	53,800 cm²
iy=	TAB("steel/"Typ; iy; Name=Profil)	=	12,50 cm
i_z=	TAB("steel/"Typ; iz; Name=Profil)	=	3,35 cm
W_{el}=	TAB("steel/"Typ; Wy; Name=Profil)	=	557,00 cm³
W_{pl} =	$a_{pll} * W_{el}$	=	634,98 cm³

Section classification As in Table 5.3.1:

Web:

$$\frac{h_1}{s * 33 * e} = 1{,}06 \sim 1$$

Þ Section class 1.

Flange:

$$\frac{b}{2 * t * 10 * e} = 0{,}70 < 1$$

Þ Section class 1.

Check bending moment strength:

$$M_{Ed} = q_d * \frac{l^2}{8} + P_d * \frac{l}{4} = 67{,}50 \text{ kNm}$$

$$M_{pl,y,Rd} = W_{pl} * \frac{f_y}{g_M * 10^3} = 135{,}65 \text{ kNm}$$

$$N_{pl,Rd} = A * \frac{f_y}{g_M * 10} = 1149{,}36 \text{ kN}$$

$$n = \frac{N_d}{N_{pl,Rd}} = 0{,}238$$

$$a = \frac{A - 2 * b * \frac{t}{10^2}}{A} = 0{,}403 < 0{,}5$$

Reduced limit moment due to tension force:

$$M_{N,y,Rd} = M_{pl,y,Rd} * \frac{1 - n}{1 - 0{,}5 * a} = 129{,}45 \text{ kNm}$$

$$\frac{M_{Ed}}{M_{N,y,Rd}} = \underline{\mathbf{0{,}52 \text{ £ } 1}}$$

Check shear strength:

$V_{Ed}= \dfrac{P_d + q_d * l}{2}$ = 37,50 kN

$A_v= 1{,}04 * h * \dfrac{s}{10^2}$ = 22,15 cm²

$V_{pl,Rd} = A_v * \dfrac{f_y}{\sqrt{3} * \gamma_M * 10}$ = 273,20 kN

$\dfrac{V_{Ed}}{V_{pl,Rd}}$ = **0,14 < 1**

Where max M occurs::

$V_S= \dfrac{P_d}{2}$ = 7,50 kN

$\dfrac{V_S}{0{,}5 * V_{pl,Rd}}$ = **0,05 $\leq$ 1**

$\Rightarrow$ No interaction!

Check buckling strength:

$\lambda_y= 100 * \dfrac{l}{i_y}$ = 48,00 < 1

$\lambda_{trans,y}= \dfrac{\lambda_y}{\pi * \sqrt{\dfrac{E_s}{f_y}}}$ = 0,511

$\lambda_z= 100 * \dfrac{l}{i_z}$ = 179,10

$\lambda_{trans,z}= \dfrac{\lambda_z}{\pi * \sqrt{\dfrac{E_s}{f_y}}}$ = 1,91

h/b = 2,00 > 1,2

t/10 = 1,07 < 4

Apply strut curve:

line =	SEL("steel/buck"; line;)	=	a
α =	TAB("steel/buck"; α; line=line)	=	0,210
χ_y =	TAB("steel/buck"; χ; $\alpha=\alpha$; λq=$\lambda_{trans,y}$)	=	0,9205

Auxilliary values:

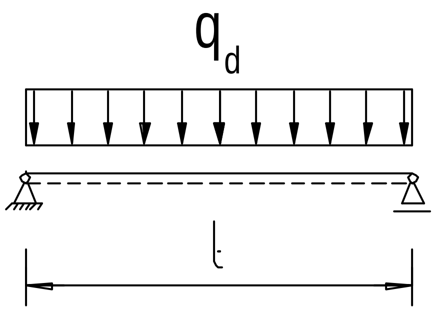
b_M= 1,30 Picture 5,5,3

m_y= $l_{trans,y} * (2 * b_M - 4) + \dfrac{W_{pl} - W_{el}}{W_{el}}$ = -0,575 < 0,9

k_y= $1 - \dfrac{m_y * N_d}{c_y * A * f_y}$ = 1,013 < 1,5

$$\frac{N_d}{c_y * A * \frac{f_y}{g_M}} + 100 * k_y * \frac{M_{Ed}}{W_{pl} * \frac{f_y}{g_M}}$$ = **0,076 < 1**

Pos.: Single-span beam:



Span length l = 7,50 m

Loads:

Dead weight g =	3,00 kN/m
Live load q =	5,00 kN/m
Snow load s_0 =	2,00 kN/m

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s=	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
f_y=	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
$\varepsilon =$	$\sqrt{\frac{235}{f_y}}$	=	1,00

Partial safety factors:

g_{fg}=	1,35
g_{fq}=	1,50
y =	0,90
g_{Mo}=	1,10

Design loads:

q_{d1}=	g_{fg}*g+g_{fq}*q	=	11,55 kN/m
q_{d2}=	g_{fg}*g+ y *g_{fq}*(q+s_0)	=	13,50 kN/m
q_d=	MAX(q_{d1};q_{d2})	=	13,50 kN/m

Design moment:

M_{Ed}=	$\frac{q_d * l^2}{8}$	=	94,92 kNm
Profil Typ =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil =	SEL("steel/"Typ; Name; Mplyd³ M_{Ed})	=	IPE 270
with $M_{pl,yd}$ =	TAB("steel/"Typ; Mplyd; Name=Profil)	=	107,00 kNm

$\frac{M_{Ed}}{M_{pl,yd}}$ = **<u>0,89 < 1</u>**

Section classification:

Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	270,00 mm
Depth of web h_1 =	TAB("steel/"Typ; h1; Name=Profil)	=	219,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	6,60 mm
Flange width b =	TAB("steel/"Typ; b; Name=Profil)	=	135,00 mm
Flange thickness t =	TAB("steel/"Typ; t; Name=Profil)	=	10,20 mm
Moment of inertia I =	TAB("steel/"Typ; Iy; Name=Profil)	=	5790,00 cm^4

Section classification as in Table 5.3.1:

Web:

$$\frac{\frac{h_1}{s}}{72 * e} = \mathbf{0{,}46 < 1}$$

Flange:

$$\frac{\frac{b}{2 * t}}{10 * e} = \mathbf{0{,}66 < 1}$$

Þ Section class 1.

Check shear strength:

V_{Ed} = $0{,}5 * q_d * l$ = 50,63 kN

A_v = $1{,}04 * h * s / 100$ = 18,53 cm²

$$V_{pl,Rd} = A_v * \frac{f_y}{\ddot{O}3 * g_{Mo} * 10} = 228{,}55 \text{ kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \underline{\mathbf{0{,}22 \text{ £} 1}}$$

Analysis working / servicing capacity as in Table 4.1:

q_{ser} = $g + y * (q + s_0)$ = 9,30 kN/m

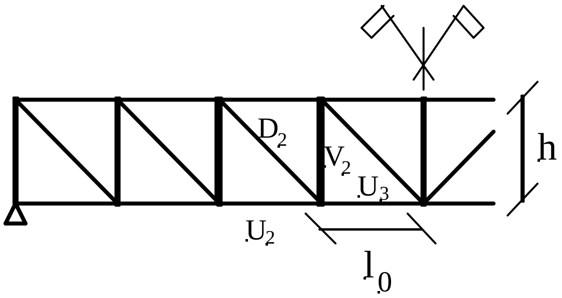
$$d_{max} = \frac{5}{384} * \frac{q_{ser} * (l * 100)^4}{E_s * I * 10} = 3{,}15 \text{ cm}$$

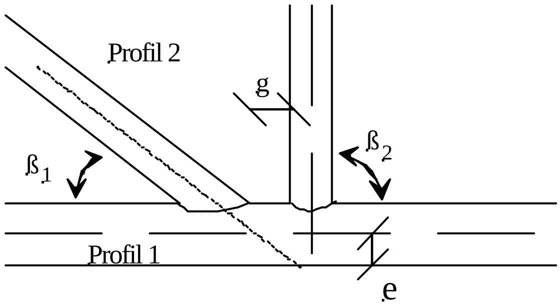
$$\frac{d_{max}}{l * \frac{100}{250}} = \underline{\mathbf{1{,}05 \text{ £} 1}}$$

$$d_2 = \frac{5}{384} * \frac{(q + s_0) * (l * 100)^4}{E_s * I * 10} = 2{,}37 \text{ cm}$$

$$\frac{d_2}{l * \frac{100}{300}} = \underline{\mathbf{0{,}95 \text{ £} 1}}$$

Lattice girder with circular hollow sectionsn:





Plan and elevation values:

Span length l_0 =	250,00 cm
b_1 =	30,96 °
b_2 =	90,00 °
h =	150,00 cm
g =	2,00 cm
e =	0,81 cm
Weld thickness a =	3,00 mm

Profil1:	SEL("steel/CHS"; Name;)	=	CHS 114.3x5
d_U =	TAB("steel/CHS"; d; Name=Profil1)	=	114,30 mm
t_U =	TAB("steel/CHS"; t; Name=Profil1)	=	5,00 mm
A_U =	TAB("steel/CHS"; A; Name=Profil1)	=	17,20 cm²

Profil2:	SEL("steel/CHS"; Name;)	=	CHS 60.3x3
d_2 =	TAB("steel/CHS"; d; Name=Profil2)	=	60,30 mm
t_2 =	TAB("steel/CHS"; t; Name=Profil2)	=	3,00 mm
A_2 =	TAB("steel/CHS"; A; Name=Profil2)	=	5,40 cm²
i_2 =	TAB("steel/CHS"; i; Name=Profil2)	=	2,03 cm

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
ε =	$\sqrt{\frac{235}{f_y}}$	=	1,00
g_M =			1,10
l_1 =	93,90 * e	=	93,90

Member forces:

U_2 =	116,67 kN
U_3 =	200,00 kN
D_2 =	97,18 kN
V_2 =	50,00 kN

$$\frac{l_0 * 10}{d_U} = 21,87 > 6$$

$$\frac{h * 10}{d_2} = 24,88 > 6$$

Section classification:

Member V2:

$$\frac{d_2}{t_2 * 50 * e^2} = \underline{0{,}40 < 1}$$

Section class 1

Analysis of the members:

Member V_2:

Section class 1 $b_A =$ 1,00

$l =$ $0{,}75*h/i_2$ = 55,42

$$l_{trans} = \frac{l}{l_1 * e} = 0{,}590$$

Apply strut curve a

$a =$ 0,21

$$j = 0{,}5 * (1 + a * (l_{trans} - 0{,}2) + l_{trans}{}^2) = 0{,}715$$

$$c = \frac{1}{j + Ö\overline{j^2 - l_{trans}{}^2}} = 0{,}8937$$

$$N_{b,Rd} = c * b_A * A_2 * \frac{f_y}{g_M * 10} = 103{,}10 \text{ kN}$$

$$\frac{V_2}{N_{b,Rd}} = \underline{0{,}485 < 1}$$

Member D2:

$$N_{Rd} = A_2 * \frac{f_y}{g_M * 10} = 115{,}36 \text{ kN}$$

$$\frac{D_2}{N_{Rd}} = \underline{0{,}842 < 1}$$

Member U2;U3

$$N_{Rd} = A_U * \frac{f_y}{g_M * 10} = 367{,}45 \text{ kN}$$

$$\frac{U_3}{N_{Rd}} = \underline{0{,}544 < 1}$$

Analysis of joint:

Check permissible stress As in Table: K.5

d_2 / d_U	=	0,528 < 1
d_2 / d_U	=	0,528 > 1
d_U / t_U	=	22,860 > 10
d_U / t_U	=	22,860 < 50
d_2 / t_2	=	20,100 > 10
d_2 / t_2	=	20,100 < 50
$(t_U + t_2) / g$	=	4,000 < 1

Analysis as given in Table. K.6:

$t_2 / 2{,}5$		=	1,200 > 1
$t_U / 25$		=	0,200 < 1
b =	MIN(b_1; b_2)	=	30,960 °
30 / b		=	0,969 < 1
$e / (d_U / 4)$		=	0,028 < 1

Þ Eccentricities can be neglected.

$a / (\mathrm{MIN}(t_U; t_2))$	=	1,000 > 0,84

Calculate maximum stress As in Table K.6

Collapse of top chord:

$$g = \frac{d_U}{2 * t_U} = 11{,}43$$

$$10 * \frac{g}{t_U} = 4{,}00$$

$$k_g = g^{0,2} * \left(1 + 0{,}024 * \frac{g^{1,2}}{1 + 2{,}7183^{\left(0{,}5 * 10 * \frac{g}{t_U} - 1{,}33\right)}}\right) = 1{,}874$$

$k_p =$ 1,00 (tension)

$$b = \frac{d_2}{d_U} = 0{,}528$$

$$N_{1,Rd} = \frac{k_g * k_p * f_y * t_U^2}{(1 - b) * \sin(b_1)} * \left(1{,}8 + 10{,}2 * \frac{d_2}{d_U}\right) * \frac{1{,}1}{g_M * 10^3} = 325{,}61 \text{ kN}$$

$$N_{2,Rd} = \frac{\sin(b_1)}{\sin(b_2)} * N_{1,Rd} = 167{,}51 \text{ kN}$$

$$\frac{V_2}{N_{2,Rd}} = \underline{\mathbf{0{,}30 < 1}}$$

Collapse due to punching shear:aus Pos. :

$$\frac{d_2}{d_U - 2 * t_U} = 0,58 < 1$$

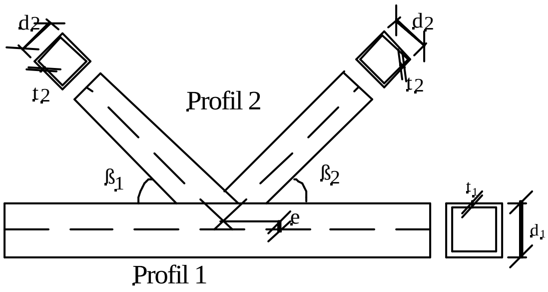
$$N_{i1,Rd} = \frac{f_y}{\ddot{O}3} * t_2 * p * d_2 * \frac{1 + \sin(b_1)}{2 * \sin(b_1)^2} * \frac{1,1}{g_M * 10^3} = 220,62\ kN$$

$D_2 / N_{i1,Rd}$ = **0,44 < 1**

$$N_{i1,Rd} = \frac{f_y}{\ddot{O}3} * t_2 * p * d_2 * \frac{1 + \sin(b_2)}{2 * \sin(b_2)^2} * \frac{1,1}{g_M * 10^3} = 77,11\ kN$$

$V_2 / N_{i1,Rd}$ = **0,65 < 1**

Truss joint with rectangular hollow sections:



Plan and elevation values:

Span length l_2 =	200,00 cm
Projection length l_{ov}=	40,00 %
b_1 =	50,00 °
b_2 =	50,00 °
e =	-2,00 cm
Weld thickness a =	3,50 mm

Profil1:	SEL("steel/SHS"; Name;)	=	SHS 100x4
d_1 =	TAB("steel/SHS"; a; Name=Profil1)	=	100,00 mm
t_1 =	TAB("steel/SHS"; t; Name=Profil1)	=	4,00 mm
A_1 =	TAB("steel/SHS"; A; Name=Profil1)	=	15,20 cm²

Profil2:	SEL("steel/SHS"; Name;)	=	SHS 60x4
d_2 =	TAB("steel/SHS"; a; Name=Profil2)	=	60,00 mm
t_2 =	TAB("steel/SHS"; t; Name=Profil2)	=	4,00 mm
A_2 =	TAB("steel/SHS"; A; Name=Profil2)	=	8,79 cm²
i_2 =	TAB("steel/SHS"; i; Name=Profil2)	=	2,27 cm

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
ε =	$\sqrt{\frac{235}{f_y}}$	=	1,00
g_M =			1,10
l_1 =	93,90 * e	=	93,90

Member forces:

U_l =	250,00 kN
U_r =	150,00 kN
D_l =	77,80 kN (compression)
D_r =	77,80 kN

Section classification:

Member Dl:

$$\frac{d_2}{t_2 * 33 * \varepsilon^2} = \underline{\mathbf{0{,}45 < 1}}$$

Section class 1

Analysis of the members:

Member V2:

$$\lambda = 0{,}75 * \frac{l_2}{i_2} = 66{,}08$$

section class 1 $\beta_A =$ 1,00

$$\lambda_{trans} = \frac{\lambda}{\lambda_1} * \sqrt{\beta_A} = 0{,}70$$

Knickspannungslinie a

$\alpha =$ 0,21

$$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans} - 0{,}2) + \lambda_{trans}^2) = 0{,}797$$

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{trans}^2}} = 0{,}8489$$

$$N_{b,Rd} = \chi * \beta_A * A_2 * \frac{f_y}{10 * \gamma_M} = 159{,}412 \text{ kN}$$

$$\frac{D_l}{N_{b,Rd}} = \underline{\mathbf{0{,}488 < 1}}$$

Member Dr:

$$N_{Rd} = A_2 * \frac{f_y}{10 * \gamma_M} = 187{,}786 \text{ kN}$$

$$\frac{D_r}{N_{Rd}} = \underline{\mathbf{0{,}414 < 1}}$$

Member U2;U3

$$N_{Rd} = A_1 * \frac{f_y}{10 * \gamma_M} = 324{,}727 \text{ kN}$$

$$\frac{U_l}{N_{Rd}} = \underline{\mathbf{0{,}770 < 1}}$$

Analysis of joint:

Check permissible stress as in Table: K.14

$\frac{d_2}{d_1}$ = 0,60 > 0,25

$\frac{d_2}{t_2 * 1{,}1 * \sqrt{\frac{E_s}{f_y}}}$ = 0,456 < 1 (compression)

$\frac{d_2}{t_2}$ = 15,000 < 35 (tension)

$\frac{d_1}{t_1}$ = 25,000 < 35 (tension)

$\frac{l_{ov}}{100}$ = 0,400 < 1

$\frac{25}{l_{ov}}$ = 0,625 < 1

Analysis as given in Table. K.12:

$t_2 / 2{,}5$ = 1,600 > 1

$t_2 / 25$ = 0,160 < 1

b = MIN(b_1; b_2) = 50,000 °

30 / b = 0,600 < 1

$\frac{e}{-d_1 * 0{,}55}$ = 0,036 < 1

Þ Eccentricities can be neglected.

a / MIN(t_1; t_2) = 0,875 > 0,84

Calculate maximum stress As in Table K.14

Only the overlapping diagonals will be analysed:

$b_{eff} = \frac{t_1}{d_1} * \frac{t_2}{t_1} * d_2$ = 2,40 cm

b_{eff} / (MIN(d_1; d_2)/10) = 0,40 < 1

$b_{e,ov} = \frac{t_2}{d_2} * \frac{t_1}{t_2} * d_2$ = 4,00 cm

$b_{e,ov}$ / (MIN(d_1; d_2)/10) = 0,67 < 1

k_n= 1,00 (tension)

so daß

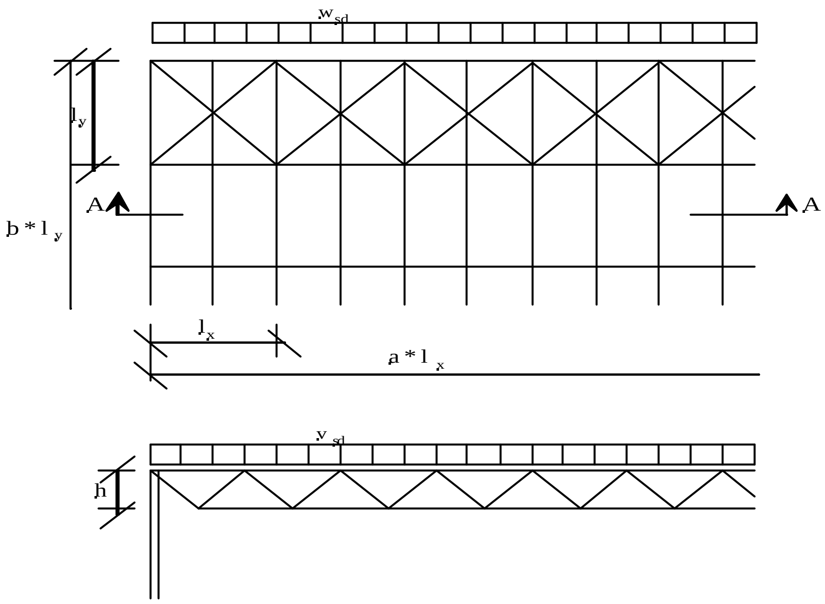
$\frac{l_{ov}}{50}$ = 0,800 < 1

$\frac{25}{l_{ov}}$ = 0,625 < 1

$N_{2,Rd} = \frac{f_y}{10^2} * t_2 * \left(\frac{l_{ov}}{50} * \frac{2 * d_2 - 4 * t_2}{10} + b_{eff} + b_{e,ov}\right)$ = 138,368 kN

$\frac{D_r}{N_{2,Rd}}$ = **<u>0,562 < 1</u>**

Roof truss:



Plan and elevation values:

Span width l_x =			3,00 m
Span length l_y =			5,00 m
Number of spans in X a =			10
Number of spans in Y b =			10
Truss depth h =			2,00 m
section for chords:			
Profil Typ =	SEL("steel/Profils"; Name;)	=	HEA
Selected Profil =	SEL("steel/"Typ; Name;)	=	HEA 140
Cross-sectional area A =	TAB("steel/"Typ; A; Name=Profil)	=	31,40 cm²
steel =	SEL("steel/EC"; Name;)	=	Fe 360
E_s =	TAB("steel/EC"; E; Name=steel)	=	210000,00 N/mm²

Loads:

w_{Ed} =	2,00 kN/m
v_{Ed} =	5,00 kN/m

Design loads:

$M_{Ed} = \quad v_{Ed} * \frac{(a * l_x)^2}{8} \quad = \quad 562{,}50$ kNm

Compression force in the top chord:

$N = \quad \frac{M_{Ed}}{h} \quad = \quad 281{,}25$ kN

Total compression force / Stabiilising force:

$N_{ges} = \quad \frac{a+1}{2} * N \quad = \quad 1546{,}88$ kN

Imperfection:

$k_{r1} = \quad \sqrt{0{,}2 + \frac{1}{\left(\frac{a+1}{2}\right)}} \quad = \quad 0{,}62$

$k_{r2} = \quad 1{,}00$

$k_r = \quad MIN(k_{r1}; k_{r2}) \quad = \quad 0{,}62 < 1$

$e_0 = \quad k_r * a * \frac{l_x}{5} \quad = \quad 3{,}72$ cm

Equivalent load instead of buckling:

$\delta_q = \quad a * \frac{l_x}{25} \quad = \quad 1{,}20$ cm

$q = \quad \frac{N_{ges}}{60 * a * l_x} * (k_r + 0{,}2) \quad = \quad 0{,}70$ kN/m

$h_{Ed} = \quad w_{Ed} + q \quad = \quad 2{,}70$ kN/m

Check deformation δq:

$E'l_{eff} = \quad 10^3 * E_s * 0{,}5 * A * l_y^2 \quad = \quad 8{,}24*10^{10}$ kNcm²

Deformation in the middle of truss:

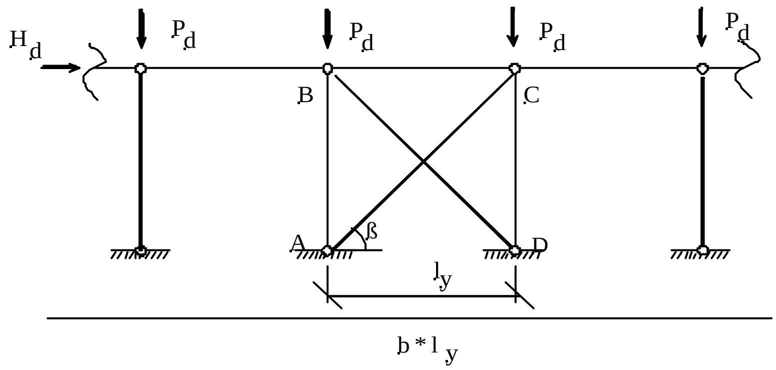
$\delta_{q1} = \quad \frac{5}{384} * h_{Ed} * \frac{(a * l_x)^4}{E'l_{eff} * 10^{-6}} \quad = \quad 0{,}35$ cm

$\frac{\delta_{q1}}{\delta_q} \quad = \quad \underline{0{,}29 < 1}$

With it the acceptance, that $\delta_q \leq l / 2500$ properly.

It can be calculated with h_{sd}.

Vertical bracing Non sway frame:



Plan and elevation values:

Frame width l_y =			6,00 m
Number of frames b =			5
Frame depth h =			5,00 m
Angle of bracing b =	$\mathrm{atan}\left(\frac{h}{l_y}\right)$	=	39,81 °
Number of bracings n =			1
Number of storeys n_s =			1
Number of column n_c =	b+1	=	6

Diagonalen wirken auf Zug und auf Druck, Or else sd=1

s_d = 2

Profil Typ1 =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil1 =	SEL("steel/"Typ1; Name;)	=	IPE 200
Moment of inertia I_{y1} =	TAB("steel/"Typ1; Iy; Name=Profil1)	=	1940,00 cm⁴
Profil Typ2 =	SEL("steel/Profils"; Name;)	=	HEA
Selected Profil2 =	SEL("steel/"Typ2; Name;)	=	HEA 180
Flange width b_t =	TAB("steel/"Typ2; b; Name=Profil2)	=	180,00 mm
Column height h_t =	TAB("steel/"Typ2; h; Name=Profil2)	=	171,00 mm
Flange thickness t =	TAB("steel/"Typ2; t; Name=Profil2)	=	9,50 mm
Moment of inertia I_{y2} =	TAB("steel/"Typ2; Iy; Name=Profil2)	=	2510,00 cm⁴
Cross-sectional area A_t =	TAB("steel/"Typ2; A; Name=Profil2)	=	45,300 cm²
i_y =	TAB("steel/"Typ2; iy; Name=Profil2)	=	7,45 cm
i_z =	TAB("steel/"Typ2; iz; Name=Profil2)	=	4,52 cm
ProfilR:	SEL("steel/CHS"; Name;)	=	CHS 168.3x6
d =	TAB("steel/CHS"; d; Name=ProfilR)	=	168,30 mm
t_r =	TAB("steel/CHS"; t; Name=ProfilR)	=	6,00 mm
A_r =	TAB("steel/CHS"; A; Name=ProfilR)	=	30,60 cm²
i_r =	TAB("steel/CHS"; i; Name=ProfilR)	=	5,74 cm
Span length l_r =	$\sqrt{h^2 + l_y^2}$	=	7,81 m

Loads:

H_d = 80,00 kN

P_d = 300,00 kN

Materials and stresses:

steel = SEL("steel/EC"; Name;) = Fe 360

E_s = TAB("steel/EC"; E; Name=steel) = 210000,00 N/mm²

f_y = TAB("steel/EC"; fy; Name=steel) = 235,00 N/mm²

$\varepsilon = \sqrt{\frac{235}{f_y}}$ = 1,00

γ_M = 1,10

λ_1 = 93,90 * ε = 93,90

Check swaying:

$k = \frac{I_{y1}}{I_{y2}} * \frac{h}{l_y}$ = 0,644

Deflection at top of frame due to horizontal load:

H = 1,00 kN

$\delta_F = \frac{(100 * h)^3 * 10}{12 * E_s * I_{y2}} * \frac{2 * k + 1}{k} * H$ = 0,702 cm

Shear strength:

$S_v = \frac{2}{10} * n * E_s * A_r * h * \frac{l_y^2}{l_r^3}$ = 485,6*10³ kN

$\delta_W = H * \frac{h}{S_v} * 100$ = 1,03*10-3 cm

Deflection at top of frame:

H = 1,00 kN

$\delta_{tot} = \frac{1}{\frac{1}{\delta_F} + \frac{1}{\delta_W}}$ = 1,028*10-3 cm

Reduction due to bracing:

$\left(1 - \frac{\delta_{tot}}{\delta_F}\right) * 100$ = 99,85 % > 80

$\Rightarrow$ Non sway frame.

Analysis of vertical bracing

Imperfection to 5.2.4.3:

$\phi_0 = \dfrac{1}{200}$ = 0,005

$k_{c1} = \sqrt{0{,}5 + \dfrac{1}{n_c}}$ = 0,82

$k_{c2} =$ 1,00

$k_c =$ MIN(k_{c1}; k_{c2}) = 0,82

$k_{s1} = \sqrt{0{,}2 + \dfrac{1}{n_s}}$ = 1,10

$k_{s2} =$ 1,00

$k_s =$ MIN(k_{s1}; k_{s2}) = 1,00

$\phi = \phi_0 * k_c * k_s$ = 0,0041

$\Delta H_d = \phi * P_d * (b + 1)$ = 7,38 kN

$H_{Ed} = H_d + \Delta H_d$ = 87,38 kN

Bracing is restrained out of plane of frame

$\dfrac{d_w}{h * 100} * \dfrac{(b+1) * P_d}{H}$ = 0,004 < 0,1

$\Rightarrow$ restrained from moving out of plane.

Analysis of lacing angles:

Design compression load:

$N_{Ed} = \dfrac{H_{Ed}}{2 * \cos(\beta)}$ = 56,88 kN

Slenderness:

$\lambda = 100 * \dfrac{l_r}{i_r}$ = 136,06

Effective slenderness:

$\lambda_{trans} = \dfrac{\lambda}{\lambda_1 * \varepsilon}$ = 1,45

Apply strut curve a:

$\alpha =$ 0,21

$\beta_A =$ 1,000

$\phi = 0{,}5 * (1 + \alpha * (\lambda_{trans} - 0{,}2) + \lambda_{trans}^2)$ = 1,683

$\chi = \dfrac{1}{\phi + \sqrt{\phi^2 - \lambda_{trans}^2}}$ = 0,3941

$N_{Rd} = \chi * \beta_A * A_r * \dfrac{f_y}{\gamma_M * 10}$ = 257,63 kN

$\dfrac{N_{Ed}}{N_{Rd}}$ = **0,221 < 1**

Analysis of frame:

Column CD:

$N_{Ed} = P_d + \frac{H_{Ed}}{2} * \frac{h}{l_y}$ = 336,41 kN

Buckling in the plane of frame:

$k_c = \frac{I_{y2}}{h * 100}$ = 5,02 cm³

$k_1 = 1{,}0 * \frac{I_{y1}}{l_y * 100}$ = 3,23 cm³

$k_2 =$ 0,0

$\eta_2 =$ 1,0

$\eta_1 = \frac{k_c}{k_c + k_1 + k_2}$ = 0,61

Slenderness ratio as in Fig. E.2.1.:

$l =$ h * 0,85 = 4,25 m

$\lambda_y = 100 * \frac{l}{i_y}$ = 57,05

$\lambda_{trans} = \frac{\lambda_y}{\lambda_1 * e}$ = 0,61

$\frac{h_t}{b_t}$ = 0,95 < 1,2

Apply strut curve b:

$\alpha =$ 0,34

$\beta_A =$ 1,000

$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans} - 0{,}2) + \lambda_{trans}^2)$ = 0,756

$\chi_y = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{trans}^2}}$ = 0,8315

Buckling outside the plane of frame

$\lambda_y = 100 * \frac{h}{i_z}$ = 110,62

$\lambda_{trans} = \frac{\lambda_y}{\lambda_1 * e}$ = 1,18

Apply strut curve c:

α = 0,49

β_A = 1,000

φ = $0{,}5 * (1 + \alpha * (\lambda_{trans} - 0{,}2) + \lambda_{trans}^2)$ = 1,436

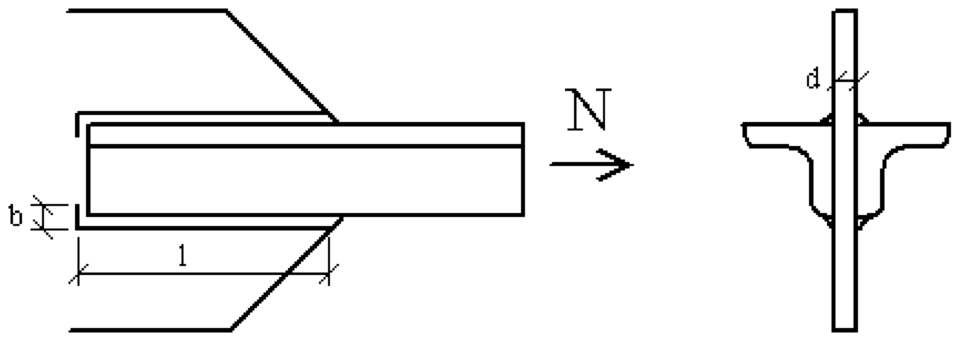
χ_z = $\dfrac{1}{\varphi + \sqrt{\varphi^2 - \lambda_{trans}^2}}$ = 0,4436

χ_{min} = $MIN(\chi_{y;}\ \chi_z)$ = 0,4436

N_{Rd} = $\chi_{min} * \beta_A * A_t * \dfrac{f_y}{\gamma_M * 10}$ = 429,30 kN

$\dfrac{N_{Ed}}{N_{Rd}}$ = **0,784 < 1**

Angle – gusset joint:



Dimensions of connection:

As demanded from EC3 6.6.2.2. b =	12,00 mm
Weld length l =	100,00 mm
Joint plate thickness d =	10,00 mm
Weld thickness a_w =	4,00 mm

Angle section P =	SEL("steel/WG"; Name;)	=	L 60x6
Area of angle A =	TAB("steel/WG"; A; Name=P)	=	6,91 cm²
Width of angle h =	TAB("steel/WG"; a; Name=P)	=	60,00 mm

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
β_w=	TAB("steel/EC"; βw; Name=steel)	=	0,80
γ_{M0} =	1,10		
γ_{Mw} =	1,25		

Plastic tension force:

$$N_{Rdp} = 2 * A * \frac{f_y}{\gamma_{M0} * 10} = 295,25 \text{ kN}$$

Maximum strength of weld:

as in 6.6.5.2(2)

$3 / a_w$	=	0,75 < 1
$MIN(40; 6 * a_w) / l$	=	0,24 < 1
$l / (150 * a_w)$	=	0,17 < 1
$12 / b$	=	1,00 < 1

Reduction factor

$$\beta_{wl1} = 1{,}2 - 0{,}2 * \frac{l}{150 * a_w} = 1{,}17$$

$$\beta_w = MIN(1; \beta_{wl1}) = 1{,}00$$

$$l = \beta_w * l = 100{,}00 \text{ mm}$$

$$N_{w,Rd} = 4 * l * a_w * \frac{f_u}{\sqrt{3} * \beta_w * \gamma_{Mw} * 10^3} = 266{,}04 \text{ kN}$$

Maximum strength of gusset:

Assumption: Stress spreads at 30° angle in gusset:

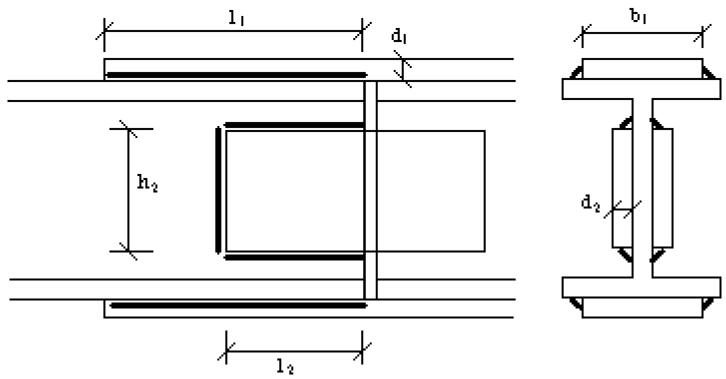
$$A = (2 * l * TAN(30) + h) * \frac{d}{100} = 17{,}55 \text{ cm}^2$$

$$N_{Rd} = A * \frac{f_y}{\gamma_{M0} * 10} = 374{,}93 \text{ kN}$$

Limit design force for tension :

$MIN(N_{Rd}; N_{w,Rd}; N_{Rdp})$ = **266,04 kN**

Welded beam splice



Dimensions of connection:

Plate thickness d_1=	20,00 mm
Plate thickness d_2=	8,00 mm
Plate width b_1=	160,00 mm
Plate thickness h_2=	280,00 mm
Weld length l_1=	240,00 mm
Weld length l_2=	200,00 mm

Welds

Weld thickness Flange a_{wf} =			6,00 mm
Weld thickness Web a_{ws} =			4,00 mm
l_{wf} =	$2 * l_1$	=	480,00 mm
l_{ws} =	$2 * l_2 + h_2$	=	680,00 mm

Profil: IPE 400

Profil Typ =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil =	SEL("steel/"Typ; Name;)	=	IPE 400
Column height h =	TAB("steel/"Typ; h; Name=Profil)	=	400,00 mm
Web thickness s =	TAB("steel/"Typ; s; Name=Profil)	=	8,60 mm
Flange width b =	TAB("steel/"Typ; b; Name=Profil)	=	180,00 mm
Flange thickness t =	TAB("steel/"Typ; t; Name=Profil)	=	13,50 mm
Cross-sectional area A =	TAB("steel/"Typ; A; Name=Profil)	=	84,50 mm
W_{el} =	TAB("steel/"Typ; Wy; Name=Profil)	=	1160,00 cm³
W_{pl} =	1,14* W_{el}	=	1322,40 cm³

Material and Partial safety factors:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
β_w=	TAB("steel/EC"; βw; Name=steel)	=	0,80
γ_{M0} =	1,10		
γ_{M2} =	1,25		
γ_{Mw} =	1,25		

Loads :

$M_{Ed} =$ 157,95 kNm

$V_{Ed} =$ 60,75 kN

Check beam:

$$M_{c,Rd} = \frac{W_{pl} * f_y}{\gamma_{M0} * 10^3} = 282{,}51 \text{ kNm}$$

$$\frac{M_{Ed}}{M_{c,Rd}} = \underline{\mathbf{0{,}56 < 1}}$$

$$A_v = 1{,}04 * \frac{h * s}{100} = 35{,}78 \text{ cm}^2$$

$$V_{pl,Rd} = A_v * \frac{f_y}{\sqrt{3} * \gamma_{M0} * 10} = 441{,}32 \text{ kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \underline{\mathbf{0{,}14 < 1}}$$

Check welds in flanges:

$$I_w = \frac{2}{12} * \frac{d_2 * h_2^{\ 3}}{10^4} = 2926{,}93 \text{ cm}^4$$

$$I_f = \frac{2 * b_1 * d_1 * \left(\frac{h + d_1}{2}\right)^2}{10^4} = 28224{,}00 \text{ cm}^4$$

$$M_w = M_{Ed} * \frac{I_w}{I_w + I_f} = 14{,}84 \text{ kNm}$$

$$M_f = M_{Ed} * \frac{I_f}{I_w + I_f} = 143{,}11 \text{ kNm}$$

Effective tension force in flange

$$N_{Ed} = \frac{M_f}{h + d_1} * 10^3 = 340{,}74 \text{ kN}$$

Limit shear stress of flange plates:

$$f_{v,wd} = \frac{f_u}{\sqrt{3} * \beta_w * \gamma_{Mw} * 10} = 20{,}78 \text{ kN/cm}^2$$

Design stress:

$$f_{Ed} = \frac{N_{Ed}}{l_{wf} * a_{wf}} * 10^2 = 11{,}83 \text{ kN/cm}^2$$

$$\frac{f_{Ed}}{f_{v,wd}} = \underline{\mathbf{0{,}57 < 1}}$$

Change welds in web:

Centre of gravity

$$e = \frac{2 * l_2 * \frac{l_2}{2} * a_{ws}}{l_{ws} * a_{ws}} = 58{,}82 \text{ mm}$$

$$I_x = \frac{a_{ws} * \frac{h_2^{\ 3}}{12} + 2 * l_2 * \left(\frac{h_2}{2}\right)^2 * a_{ws}}{10^{\ 4}} = 3867{,}73 \text{ cm}^4$$

$$I_y = \frac{h_2 * a_{ws} * e^{\ 2} + \frac{2}{12} * a_{ws} * l_2^{\ 3} + 2 * l_2 * a_{ws} * \left(\frac{l_2}{2} - e\right)^2}{10^{\ 4}} = 1192{,}16 \text{ cm}^4$$

Design moment

$$M_{Ew} = M_w + V_{Ed} * \frac{l_2 - e}{10^{\ 3}} = 23{,}42 \text{ kNm}$$

Maximum shear stress

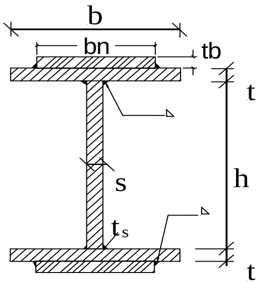
$$f_{Ex} = 10 * M_{Ew} * \frac{h_2}{4 * (I_x + I_y)} = 3{,}24 \text{ kN/cm}^2$$

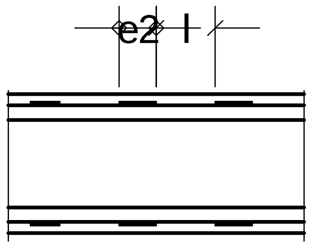
$$f_{Ey} = \frac{V_{Ed}}{l_{ws} * \frac{a_{ws}}{100}} + \frac{100 * M_{Ew} * \frac{l_2 - e}{10}}{2 * (I_x + I_y)} = 5{,}50 \text{ kN/cm}^2$$

$$f_E = \sqrt{f_{Ex}^{\ 2} + f_{Ey}^{\ 2}} = 6{,}38 \text{ kN/cm}^2$$

$$\frac{f_E}{f_{v,wd}} = \underline{\mathbf{0{,}31 < 1}}$$

Beam with cover plates on flange:





Plan and elevation values:

Flange width b =	16,00 cm
Flange thickness t =	1,20 cm
Depth of web h =	34,00 cm
Web thickness s =	0,80 cm
Plate thickness t_b =	0,80 cm
Weld thickness a =	0,30 cm
Weld distance l =	9,50 cm

Loads:

M_{Ed} =	241,90 kNm
V_{Ed} =	97,50 kN

Materials and stresses

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_y =	TAB("steel/EC"; fy; Name=steel)	=	235,00 N/mm²
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
β_w=	TAB("steel/EC"; βw; Name=steel)	=	0,80
ε =	$\sqrt{\frac{235}{f_y}}$	=	1,00

Partial safety factors:

γ_M =	1,10
γ_{Mw} =	1,25

Design:

$$W_{pl} = \left(b * t * \frac{h+t}{2} + \frac{h}{2} * s * \frac{h}{4}\right) * 2 = 907{,}04 \text{ cm}^3$$

$$M_{pl} = 0{,}001 * W_{pl} * \frac{f_y}{\gamma_M} = 193{,}78 \text{ kNm}$$

$$V_{pl,Rd} = h * s * \frac{f_y}{\sqrt{3} * \gamma_M * 10} = 335{,}49 \text{ kN}$$

Distance between cover plates:

$$b_n = \frac{\gamma_M * 10 * \frac{M_{Ed}}{f_y} - W_{pl}}{2 * t_b * \left(\frac{h}{2} + t + \frac{t_b}{2}\right)} = -30{,}10 \text{ cm}$$

gew b_n = 7,60 cm

$$W_{pl} = \left(b * t * \frac{h+t}{2} + \frac{h}{2} * s * \frac{h}{4}\right) * 2 + 2 * t_b * \left(\frac{h}{2} + t + \frac{t_b}{2}\right) * b_n \quad = \quad 1133{,}22 \text{ cm}^3$$

$$M_{pl,Rd} = 0{,}001 * W_{pl} * \frac{f_y}{\gamma_M} \quad = \quad 242{,}10 \text{ kNm}$$

Analysis:

$$\frac{M_{Ed}}{M_{pl,Rd}} = \underline{\mathbf{1{,}00 \leq 1}}$$

$$\frac{V_{Ed}}{\frac{1}{2} * V_{pl,Rd}} = \underline{\mathbf{0{,}58 \leq 1}}$$

Length e2 of weld:

$$I_y = \frac{b * (h + 2 * t)^3 - (b - s) * h^3}{12} + 2 * t_b * b_n * \left(\frac{h}{2} + t + \frac{t_b}{2}\right)^2 \quad = \quad 18726{,}53 \text{ cm}^4$$

$$T' = V_{Ed} * \frac{t_b * b_n * \left(\frac{h}{2} + t + \frac{t_b}{2}\right)}{I_y} \quad = \quad 0{,}59 \text{ kN/cm}$$

$$f_{v,wd} = \frac{f_u}{\sqrt{3} * \beta_w * \gamma_{Mw} * 10} \quad = \quad 20{,}78 \text{ kN/cm}^2$$

$$F_{w,Rd} = 0{,}2 * a * f_{v,wd} \quad = \quad 1{,}25 \text{ kN/cm}$$

$$e_2 = \frac{T}{F_{w,Rd}} \quad = \quad 0{,}47 \text{ cm}$$

sel e2 = 40,00 mm (Minimum allowed length)

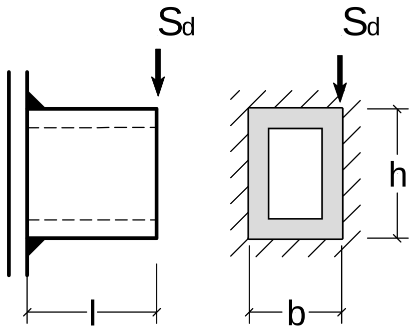
$$T = T' * (e_2 + l) \quad = \quad 29{,}20 \text{ kN}$$

$$F_{w,Rd} = 0{,}2 * a * e_2 * f_{v,wd} \quad = \quad 49{,}87 \text{ kN}$$

Analysis:

$$\frac{T}{F_{w,Rd}} = \underline{\mathbf{0{,}59 \leq 1}}$$

Welded connection



Plan and elevation values:

Span length l =	200,00 mm
Width b =	80,00 mm
Web depth h =	140,00 mm
Weld thickness a_w =	6,00 mm

Loads:

S_d = 100,00 kN

Materials and stresses:

steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_u =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
β_w=	TAB("steel/EC"; βw; Name=steel)	=	0,80

γ_{Mw} = 1,25

Design:

Weld properties:

$A = 0{,}02 * a_w * (b+h) = 26{,}40\ \text{cm}^2$

$$I_y = \frac{2 * b * a_w * \left(\frac{h}{2}\right)^2 + 2 * a_w * \frac{h^3}{12}}{10^4} = 744{,}80\ \text{cm}^4$$

$$I_z = \frac{2 * h * a_w * \left(\frac{b}{2}\right)^2 + 2 * a_w * \frac{b^3}{12}}{10^4} = 320{,}00\ \text{cm}^4$$

$I_p = I_y + I_z = 1064{,}80\ \text{cm}^4$

Design load in relation to the centroid of weld:

$S_{dz} = S_d = 100{,}00$ kN

$$M_{y,Ed} = S_d * \frac{l}{10} = 2000{,}00\ \text{kNcm}$$

$$M_{t,Ed} = S_d * \frac{b}{20} = 400{,}00\ \text{kNcm}$$

Check stress at weakest point:

$$\sigma_A = \frac{M_{y,Ed}}{I_y} * \frac{h}{20} = 18{,}80 \text{ kN/cm}^2$$

$$\tau_1 = \frac{M_{t,Ed}}{I_p} * \frac{h}{20} = 2{,}63 \text{ kN/cm}^2$$

$$\tau_2 = \frac{M_{t,Ed}}{I_p} * \frac{b}{20} + \frac{S_{dz}}{A} = 5{,}29 \text{ kN/cm}^2$$

$$\Rightarrow \sigma_{res} = \sqrt{\sigma_A^{\ 2} + \tau_1^{\ 2} + \tau_2^{\ 2}} = \underline{\mathbf{19{,}71\ kN/cm^2}}$$

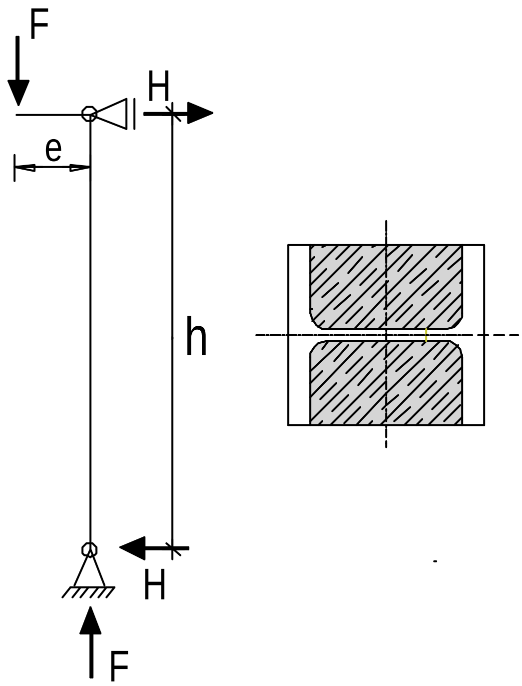
Check weld strength:

Limit shear stress

$$f_{w,Rd} = \frac{f_u}{\sqrt{3} * \beta_w * \gamma_{Mw} * 10} = 20{,}78 \text{ kN/cm}^2$$

$$\frac{\sigma_{res}}{f_{w,Rd}} = \underline{\mathbf{0{,}95 < 1}}$$

Concrete encasement composite column with eccentric force application:



System:

Height h =			8,00 m
Eccentricity e =			0,15 m
Beam type =	SEL("steel/profils"; Name;)	=	HEB
Nominal height NH =	SEL("steel/"type; NH;)	=	340
A_a =	TAB("steel/"type; A; NH=NH)	=	171,00 cm²
h_p =	TAB("steel/"type; h; NH=NH)	=	340,00 mm
s =	TAB("steel/"type; s; NH=NH)	=	12,00 mm
t =	TAB("steel/"type; t; NH=NH)	=	21,50 mm
b =	TAB("steel/"type; b; NH=NH)	=	300,00 mm
I_{ya} =	TAB("steel/"type; I_y ; NH=NH)	=	36660,00 cm⁴
W_{pa} =	1.14*TAB("steel/"type; W_y ; NH=NH)	=	2462,40 cm³
W_{pa} =			2408,00 cm³

Materials:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("steel/EC"; NameEN;)	=	S235
E_{cm} =	TAB("concrete/EC"; E_{cm}; Name=Concrete)	=	32000,00 N/mm²
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	30,00 N/mm²

The value for a for use in a contry should lie between 0,8 - 1,0 and me be found in a National Annex.
The Recommended Value for a is 1,0

a =	1,00		
E_a =	TAB("steel/EC"; E; NameEN=Steel)	=	210000,00 N/mm²
f_{yk} =	TAB("steel/EC"; fy; NameEN=Steel)	=	235,00 N/mm²

Partial safety factors:

Dead load γ_G =	1,35
Imposed Load γ_Q =	1,50
Concrete γ_c =	1,50
Construction steel γ_a =	1,10
Profile steel sheeting γ_{ap} =	1,10
Longitudinal shear γ_{vs} =	1,25
Elastic modulus g =	1,35

Load:

g_k = 450,00 kN

q_k = 1000,00 kN

Calculation:

N_{Ed} =	$g_k * g_G + q_k * g_Q$	=	2107,50 kN
M_{Ed} =	$N_{Ed} * e$	=	316,13 kNm
V_{Ed} =	M_{Ed} / h	=	39,52 kN
f_{cd} =	$a * f_{ck} / g_c$	=	20,00 N/mm²
f_{yd} =	f_{yk} / g_a	=	213,64 N/mm²
E_{cd} =	E_{cm} / g	=	23703,70 N/mm²
A_c =	$(h_p * b) *10^{-2} - A_a$	=	849,00 cm²
I_{yc} =	$(h_p^3 * b / 120000) - I_{ya}$	=	61600,00 cm⁴

Local buckling of parts of the steel section:

$$e = Ö\frac{235}{f_{yk}} \qquad = \qquad 1,00$$

$$\frac{b / t}{e * 44} \qquad = \qquad \underline{0,32 < 1}$$

Load-bearing capacity of the column under centric pressure with buckling risk:

$$N_{pl,Rd} = \frac{A_a * f_{yd} + A_c * f_{cd}}{10} \qquad = \qquad 5351,24 \text{ kN}$$

Structural verification procudure applicable?

$$d = \frac{A_a * f_{yd}}{10 * N_{pl,Rd}} \qquad = \qquad 0,68$$

$0.2 / d$ = $\underline{0,29 < 1}$

$d / 0.9$ = $\underline{0,76 < 1}$

Þ Structural verification procudure applicable.

On the effect of lateral shear:

$$A_v = (1.04 * h_p * s) * 10^{-2} \qquad = \qquad 42,43 \text{ cm}^2$$

$$V_{pl,Rd} = \frac{A_v * f_{yd}}{Ö3 * 10} \qquad = \qquad 523,35 \text{ kN}$$

$V_{Ed} / V_{pl,Rd}$ = $\underline{0,08 < 0,5}$

Þ The lateral shear force has no effect on the load-bearing capacity.

Cross section bearing capacity under pressure and single-axis bending:

Plastic limit moment at point D:

$W_{pc} = \dfrac{b * h_p^2}{4000} - W_{pa}$ = 6262,00 cm³

$M_{max,Rd} = (W_{pa} * f_{yd} + 1/2 * W_{pc} * f_{cd}) * 10^{-3}$ = 577,07 kNm

$N_D = 1/20 * A_c * f_{cd}$ = 849,00 kN

Plastic limit moment in points B and C:

$N_{pm,Rd} = (A_c * f_{cd}) * 10^{-1}$ = 1698,00 kN

$h_N = \dfrac{N_{pm,Rd}}{0{,}2 * b * 10^{-1} * f_{cd} + 2 * s * 10^{-2} * (2 * f_{yd} - f_{cd})}$ = 7,80 cm

$\dfrac{h_N}{(b / 2 - t) * 10^{-1}}$ = **<u>0,61 ≤ 1</u>**

⇒ Plastic neutral axis is in the rib!

$W_{pan} = s/10 * h_N^2$ = 73,01 cm³

$W_{pcn} = (b * h_N^2) * 10^{-1} - W_{pan}$ = 1752,19 cm³

$M_{N,Rd} = (W_{pan} * f_{yd} + 1/2 * W_{pcn} * f_{cd}) * 10^{-3}$ = 33,12 kNm

The plastic limit moment in points B and C is:

$M_{pl,Rd} = M_{max,Rd} - M_{N,Rd}$ = 543,95 kNm

$N_B =$ 0,00 kN

$N_C = 2 * N_D$ = 1698,00 kN

Load-bearing capacity of a composite column under pressure and single-axis bending:

$N_{pl,Rk} = (A_a * f_{yk} + A_c * \alpha * f_{ck}) * 10^{-1}$ = 6565,50 kN

Effective elastic bending rigidity:

$EI_e = (E_a * I_{ya} + 0.8 * E_{cd} * I_{yc}) * 10^{-5}$ = 88667,18 kN/m²

Reference degree of slenderness:

$N_{cr} = \dfrac{\pi^2 * EI_e}{h^2}$ = 13673,57 kN

$\lambda' = \sqrt{\dfrac{N_{pl,Rk}}{N_{cr}}}$ = 0,69

⇒ Long-term behaviour (creep and shrinkage) may be disregarded.

Strut curve b:

Type = SEL("comp/buck"; Desc;) = concrete-encased profile strong axis

line = TAB("comp/buck"; line; Desc=Type) = b

χ = TAB("comp/buck"; χ ; line=line; λlat=λ') = 0,789

$\chi_d = N_{Ed} / N_{pl,Rd}$ = 0,394

Proportion of edge moments r = 0,00

$\chi_N = \chi * ((1 - r) / 4)$ = 0,197

The resulting values for point C are:

$\chi_c = N_C / N_{pl,Rd}$ = 0,317

$\mu_c =$ 1,00

$\mu_k = \dfrac{1 - \chi}{1 - \chi_c} * \mu_c$ = 0,309

$$m_d = \frac{1 - k_d}{1 - k} * m_k \qquad = \qquad 0{,}887$$

The length m is:

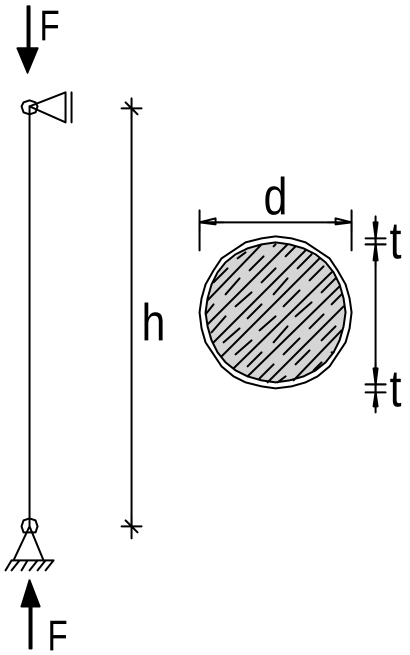
$$m = \qquad m_d - m_k * (k_d - k_N) / (k - k_N) \qquad = \qquad 0{,}784$$

Structural verification of load-bearing capacity:

$$\frac{N_{Ed}}{k * N_{pl,Rd}} \qquad = \qquad \underline{\mathbf{0{,}50\ £1}}$$

$$\frac{M_{Ed}}{0{,}9 * m * M_{pl,Rd}} \qquad = \qquad \underline{\mathbf{0{,}82\ £1}}$$

Concrete-filled hollow section:



System:

Column height h =	7,00 m
Outside pipe diameter d =	273,00 mm
Pipe thickness t =	6,30 mm

Load:

g_k =	500,00 kN
q_k =	600,00 kN

Materials:

Concrete =	SEL("concrete/EC"; Name;)	=	C30/37
Steel =	SEL("steel/EC"; NameEN;)	=	S355
E_{cm} =	TAB("concrete/EC"; E_{cm}; Name=Concrete)	=	32800,00 N/mm²
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)|	=	30,00 N/mm²
E_a =	TAB("steel/EC"; E; NameEN=Steel)	=	210000,00 N/mm²
f_{yk} =	TAB("steel/EC"; fy; NameEN=Steel)	=	355,00 N/mm²

The value for a for use in a contry should lie between 0,8 - 1,0 and me be found in a National Annex.
The Recommended Value for a is 1,00

a = 1,00

Partial safety factors:

Dead load γ_G =	1,35
Imposed Load γ_Q =	1,50
Concrete γ_c =	1,50
Construction steel γ_a =	1,10
Profile steel sheeting γ_{ap} =	1,10
Longitudinal shear γ_{vs} =	1,25
Concrete elastic modulus g =	1,35

Calculation:

$f_{yd} = f_{yk} / \gamma_a$ = 322,73 N/mm²

$f_{cd} = \alpha * f_{ck} / \gamma_c$ = 20,00 N/mm²

$N_{Ed} = \gamma_G * g_k + \gamma_Q * q_k$ = 1575,00 kN

$f_{yd} = f_{yk} / \gamma_a$ = 322,73 N/mm²

$A_a = \frac{\pi}{400} * \left(d^2 - (d - 2 * t)^2\right)$ = 52,79 cm²

$I_a = \frac{\pi}{640000} * \left(d^4 - (d - 2 * t)^4\right)$ = 4695,82 cm⁴

$E_{cd} = E_{cm} / \gamma$ = 24296,30 N/mm²

$A_c = \frac{\pi * (d - 2 * t)^2}{400}$ = 532,56 cm²

$I_c = \frac{\pi * (d - 2 * t)^4}{640000}$ = 22570,10 cm⁴

Local buckling of parts of the steel section:

$\varepsilon = \sqrt{\frac{235}{f_{yk}}}$ = 0,81

$\left(\frac{d}{t}\right) / \left(90 * \varepsilon^2\right)$ = **<u>0,73 ≤ 1</u>**

Structural verification of the column's load-bearing capacity:

$N_{pl,Rd} = (A_a * f_{yd} + A_c * f_{cd}) * 10^{-1}$ = 2768,81 kN

Structural verification procudure applicable?

$\delta = \frac{\left(A_a * \frac{f_{yk}}{\gamma_a}\right)}{N_{pl,Rd} * 10}$ = 0,62

$0.2 / \delta$ = **<u>0,32 < 1</u>**

$\delta / 0.9$ = **<u>0,69 < 1</u>**

⇒ Structural verification procudure applicable

$N_{pl,Rk} = (A_a * f_{yk} + A_c * \alpha * f_{ck}) * 10^{-1}$ = 3471,72 kN

Effective elastic bending rigidity:

$EI_e = (E_a * I_a + 0,8 * E_{cd} * I_c) * 10^{-5}$ = 14248,18 kNm²

Reference degree of slenderness:

$N_{cr} = \pi^2 * EI_e / h^2$ = 2869,87 kN

$\bar{\lambda}' = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}}$ = 1,10

$\bar{\lambda}'_g = \frac{0,8}{(1 - \delta)}$ = 2,11

l' / l'_g = 0,52 < 1

Þ Long-term behaviour (creep and shrinkage) may be disregarded.

Concrete-filled hollow section, therefore strut curve a

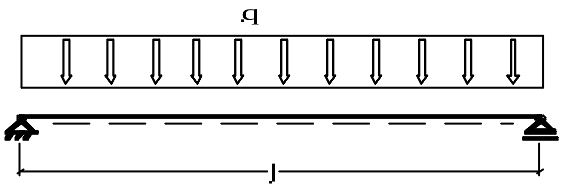
Type =	SEL("comp/buck"; Desc;)	=	concrete-filled hollow section
line =	TAB("comp/buck"; line; Desc=Type)	=	a
k =	TAB("comp/buck"; k ; line=line; llat=l')	=	0,596
N_{Rd} =	k * $N_{pl,Rd}$	=	1650,21 kN

Structural verification:

N_{Ed} / N_{Rd} = **0,95 < 1**

The ultimate load bearing capacity is almost completely exploited.

Composite beams - Ultimate load bearing capacity:



System:

Beam length L = 12,00 m
Beam distance s = 3,60 m

Materials:

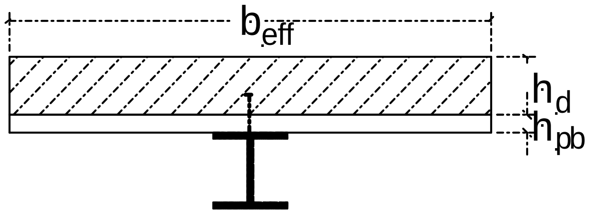
Beam type =	SEL("steel/profils"; Name;)	=	IPE
Nominal height NH =	SEL("steel/"type; NH;)	=	450
A =	TAB("steel/"type; A; NH=NH)	=	98,80 cm²
h =	TAB("steel/"type; h; NH=NH)/10	=	45,00 mm
I_y =	TAB("steel/"type; I_y ; NH=NH)	=	33740,00 cm4

Shear connectors Æ 22
h_b = 100,00 mm
d = 22,00 mm
Distance of shear connectors e_L = 15,00 cm

Concrete =	SEL("concrete/EC"; Name;)	=	C25/30
Steel =	SEL("steel/EC"; NameEN;)	=	S355

Load:

g_{1k} = 15,28 kN/m
g_{2k} = 7,74 kN/m (after removal of temporary props)
q_k = 18,00 kN/m



Floor thickness h_d = 16,00 cm
Thickness of profile steel sheeting h_{pb} = 5,10 cm

Material properties:

E_{cm} =	TAB("concrete/EC"; E_{cm}; Name=Concrete)	=	31500,00 N/mm²
E_a =	TAB("steel/EC"; E; NameEN=Steel)	=	210000,00 N/mm²

Cross section load bearing capacity at full shear connection:

b_{eff} =	2 * L / 8	=	3,00 m
b_{eff} =	MIN(b_{eff} ;s)	=	**3,00 m**

Working state analysis of composite beams:

Form factors:

Short term load at point of time t =0:

$n_0 =$ E_a / E_{cm} = 6,67

Effective component thickness:

Beam width in air u = 360,00 cm

$h_o =$ $2 * h_d * u / u$ = 32,00 cm

Final creep value t_0 =14 days

according to EC2 Appendix1 j = 2,70

constant continuous load $y_{A,B}$ = 1,10

Shrinkage $y_{A,S}$ = 0,55

Creep under continuous load nj = $n_0 * (1+ y_{A,B} * j)$ = 26,48

Shrinkage n_S = $n_0 * (1+ y_{A,S} * j)$ = 16,57

A_c = $100 * b_{eff} * (h_d - h_{pb})$ = 3270,00 cm²

I_c = $100 * b_{eff} * (h_d - h_{pb})^3 / 12$ = 32375,72 cm²m²

$$z'_{s0} = \frac{A * \left(\frac{h}{2} + h_d\right) + \frac{A_c}{n_0} * \left(\frac{h_d - h_{pb}}{2}\right)}{A + \frac{A_c}{n_0}} = 10,99 \text{ cm}$$

$$\left(I_y + \frac{I_c}{n_0}\right) * 10^{-4} + \left(A * \left(\frac{h}{2} + h_d\right)^2 + \frac{A_c}{n_0} * \left(\frac{h_d - h_{pb}}{2}\right)^2\right) * 10^{-4} = 19,96 \text{ cm}^2\text{m}^2$$

$$\left(z'_{s0} * 10^{-2}\right)^2 * \left(A + \frac{A_c}{n_0}\right) * (-1) = -7,11 \text{ cm}^2\text{m}^2$$

$\mathbf{I_{i0}}$ = **12,85 cm²m²**

$$z'_{sj} = \frac{A * \left(\frac{h}{2} + h_d\right) + \frac{A_c}{n_j} * (h_d - h_{pb}) / 2}{A + A_c / n_j} = 20,14 \text{ cm}$$

$$\left(I_y + \frac{I_c}{n_j}\right) * 10^{-4} + \left(A * \left(\frac{h}{2} + h_d\right)^2 + \frac{A_c}{n_j} * \left(\frac{h_d - h_{pb}}{2}\right)^2\right) * 10^{-4} = 18,51 \text{ cm}^2\text{m}^2$$

$$\left(z's_j * 10^{-2}\right)^2 * \left(A + \frac{A_c}{n_j}\right) * (-1) = -9,02 \text{ cm}^2\text{m}^2$$

$\mathbf{I_{iphi}}$ = **9,49 cm²m²**

I_{ij} = I_{iphi} = **9,49 cm²m²**

$$z'_{sS} = \frac{A * \left(\frac{h}{2} + h_d\right) + \frac{A_c}{n_S} * (h_d - h_{pb})/2}{A + A_c / n_S} = 16{,}48 \text{ cm}$$

$$\left(I_y + \frac{I_c}{n_S}\right) * 10^{-4} + \left(A * \left(\frac{h}{2} + h_d\right)^2 + \frac{A_c}{n_S} * \left(\frac{h_d - h_{pb}}{2}\right)^2\right) * 10^{-4} = 18{,}80 \text{ cm}^2\text{m}^2$$

$$\left(z'_{sS} * 10^{-2}\right)^2 * \left(A + \frac{A_c}{n_S}\right) * (-1) = -8{,}04 \text{ cm}^2\text{m}^2$$

I_{iS} = **10,76 cm²m²**

Calculation of midspan deflection:

Release of temporary prop at point of time t =0

B =	g_{1k} * 1.25 * L/2	=	114,60 kN
f_{B0} =	100 * B * L³ / (48 * E_a/10 * I_{i0})	=	1,53 cm

Assumption: 40% of the live load as a permanent load at point of time t =0:

Ratio=			0,40
$f_{g2,0}$ =	100 * 5/384 *(g_{2k} + Ratio* q_k) * L^4 / (E_a/10 * I_{i0})	=	1,49 cm

from imposed load (short term ratio):

Ratio2 =	1 - Ratio	=	0,60
f_q =	100 * 5/384 * Ratio2 * q_k * L^4 / (E_a/10 * I_{i0})	=	1,08 cm

From removal of temporary prop at point of time t =¥ :

f_B =	f_{B0} * I_{i0} / I_i j	=	2,07 cm

From g_{k2} + $q_{permanent}$ at point of time t =¥ :

f_{g2} =	$f_{g2,0}$ * I_{i0} / I_i j	=	2,02 cm

From shrinkage at point of time t =¥ :

Final shrinkage value e_{cS} = 325*10^{-6}

N_{sch} =	(A_c * E_a / n_S * e_{cS}) / 10	=	1346,88 kN
z_{sch} =	z'_{sS} - (h_d-h_{pb})/2	=	11,03 cm
M_{sch} =	N_{sch} * z_{sch} / 100	=	148,56 kN
f_{sch} =	100 * 1/8 * M_{sch} * L² / (E_a/10 * I_{iS})	=	1,18 cm

Maximum deflection:

max_f =	f_B + f_{g2} + f_{sch} + f_q	=	6,35 cm

Recommended camber:

f_0 =	f_{B0} + $f_{g2,0}$	=	3,02 cm

Maximum deflection in final state:

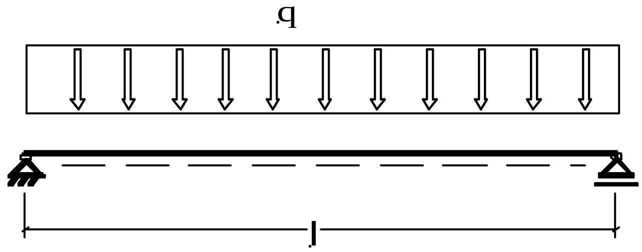
f =	max_f - f_0	=	3,33 cm

Structural verification:

f/100 / (L / 300) = **0,83 < 1**

The deflection in the working state meets the required standard!

Composite floor in the final state:



System:

Steel profile with lugs; values according to manufacturer's specifications (approval)

m =	166,00
k =	0,15
Sheet thickness t =	0,86 mm
e =	17,00 mm
Beam length L =	4,80 m
Stability f_{yp} =	350,00 N/mm²
Cross section area A_p =	1562,00 N/mm²
$t_{u,Rd}$ =	280,00 kN/m
minimum width of concrete ribs b_0 =	750,00 mm/m

Concrete:

Floor thickness h_t = 14,00 cm

Concrete =	SEL("concrete/EC"; Name;)	=	C25/30
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	25,00 N/mm²
f_{ctk005} =	TAB("concrete/EC"; fctk05; Name=Concrete)|	=	1,80 N/mm²

Influences:

Final state:

Permanent load of composite floor G_1 =	3,30 kN/m
Permanent load of design loads G_2 =	1,20 kN/m
Imposed load Q =	5,00 kN/m

Partial safety factors:

Dead load γ_G =	1,35
Imposed Load γ_Q =	1,50
Concrete γ_c =	1,50
Profile steel sheeting γ_{ap} =	1,10
Longitudinal shear γ_{vs} =	1,25

Structural verification of the composite floor in the final state:

b = 1000,00 mm

Deflection:

M_{Ed} = $(\gamma_G * (G_1 + G_2) + \gamma_Q * Q) * L^2 / 8$ = 39,10 kNm/m

Design resistance:

N_{cf} = $(A_p * f_{yp} / \gamma_{ap})/1000$ = 497,00 kN

The value for α for use in a contry should lie between 0,8 - 1,0 and me be found in a National Annex.

The Recommended Value for α is 1,0

α = 1,00

$$c = \frac{N_{cf} * 10^3}{b * \left(\alpha * \frac{f_{ck}}{\gamma_c}\right)} = 29{,}82$$

d_p = $10*h_t - e$ = 123,00 mm

$$M_{p,Rd} = N_{cf} * \frac{d_p - 0{,}5 * c}{10^3} = 53{,}72 \text{ kNm/m}$$

Structural verification:

$M_{Ed} / M_{p,Rd}$ = **0,73 < 1**

Longitudinal shear, m + k - method:

Design shear force at the footing point:

V_{Ed} = $(\gamma_G*(G_1+G_2) + \gamma_Q * Q) * L / 2$ = 32,58 kN/m

Longitudinal shear resistance:

Shear length L_s = 1000 * L / 4 = 1200,00 mm

$$V_{l,Rd} = \frac{b * d_p * \left(m * \frac{A_p}{b * L_s} + k\right)}{\gamma_{vs}} * 10^{-3} = 36{,}02 \text{ kN/m}$$

Structural verification:

$V_{Ed} / V_{l,Rd}$ = **0,90 < 1**

Longitudinal shear, partial shear connection (EC4, Appendix E):

Design value of the bond strength according to manufacturer's information:

Shear length at full shear connection η = 1,00

$$L_{sf} = \frac{N_{cf}}{b * 10^{-3} * \tau_{u,Rd}} = 1{,}77 \text{ m}$$

Design resistance:

τ_{Rd} = $0{,}09 * \sqrt[3]{f_{ck}}$ = 0,26 N/mm²

k_v = $1.6 - d_p /1000$ = 1,48 > 1

A_p = $b_0 * t$ = 645,00 mm²

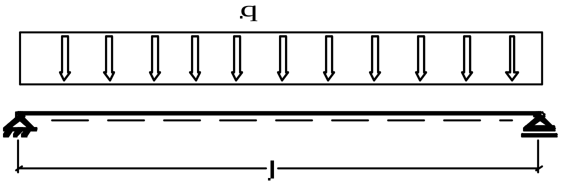
$$\rho = \frac{A_p}{b_0 * d_p} = 0{,}007$$

$V_{v,Rd}$ = $(b_0 * d_p * \tau_{Rd} * k_v * (1.2 + 40 * \rho))/1000$ = 52,54 kN/m

Structural verification:

$V_{Ed} / V_{v,Rd}$ = **0,62 < 1**

Composite beams - Ultimate load bearing capacity:



System:

Beam length L =	12,00 m
Number of temporary props a =	1,00
Beam distance s =	3,60 m
Flange width b_0 =	12,60 cm

Load:

g_{1k} =	14,50 kN/m
g_{2k} =	7,74 kN/m (Hilfsunterstützung entfernt)
q_k =	18,00 kN/m

Materials:

Concrete =	SEL("concrete/EC"; Name;)	=	C25/30
Steel =	SEL("steel/EC"; NameEN;)	=	S355
Beam type =	SEL("steel/profils"; Name;)	=	IPE
Nominal height NH =	SEL("steel/"type; NH;)	=	400
A =	TAB("steel/"type; A; NH=NH)	=	84,50 cm²
h =	TAB("steel/"type; h; NH=NH)/10	=	40,00 cm
t =	TAB("steel/"type; s; NH=NH)/10	=	0,86 cm
I_y =	TAB("steel/"type; I_y ; NH=NH)	=	23130,00 cm⁴
M_{pl} =	TAB("steel/"type; M_{plyd}; NH=NH)	=	289,00 kNm
M_{pl} =	IF(Steel="S355";1,5;1)*M_{pl}	=	433,50 kNm

Shear connectors Æ 22

h_b =	100,00 mm
d =	19,00 mm
Distance of shear connectors e_L =	15,00 cm
Ultimate strength f_u =	450,00 N/mm²

Partial safety factors:

Dead load γ_G = 1,35

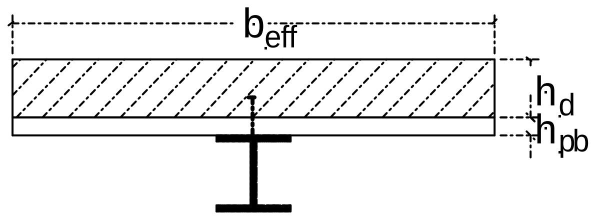
Imposed Load γ_Q = 1,50

Concrete γ_c = 1,50

Construction steel γ_a = 1,10

Profile steel sheeting γ_{ap} = 1,10

Longitudinal shear γ_{vs} = 1,25



Floor thickness h_d = 16,00 cm

Thickness of profile steel sheeting h_{pb} = 5,10 cm

Cross-sectional area of plate A_p = 15,62 kN/m

Material properties:

E_{cm} = TAB("concrete/EC"; E_{cm}; Name=Concrete) = 30500,00 N/mm²

f_{ck} = TAB("concrete/EC"; fck; Name=Concrete) = 25,00 N/mm²

f_{ctk005} = TAB("concrete/EC"; fctk05; Name=Concrete) = 1,80 N/mm²

f_{cd} = f_{ck} / g_c = 16,67 N/mm²

E_a = TAB("steel/EC"; E; NameEN=Steel) = 210000,00 N/mm²

f_{yk} = TAB("steel/EC"; fy; NameEN=Steel) = 355,00 N/mm²

f_{yd} = f_{yk} / g_a = 322,73 N/mm²

Holorib sheeting in accordance with building regulations approval:

f_{yp} = 280,00 N/mm²

Conformation of fitness for purpose:

Stress resultants in ultimate state analysis of fitness for purpose:

r_d = g_G * (g_{1k} + g_{2k}) + g_Q * q_k = 57,02 kN/m

M_{Ed} = r_d * L² / 8 = 1026,36 kNm

V_{Ed} = r_d *L / 2 = 342,12 kN

Cross section load bearing capacity with full shear connection:

b_{eff} = 2 * L / 8 = 3,00 m

b_{eff} = MIN(b_{eff} ; s) = **3,00 m**

A_v = 1.04 * h * t = 35,78 cm²

$N_{pl,a,Rd}$ = A * f_{yd} / 10 = 2727,07 kN

N_{cd} = 0,85 * f_{cd} * 100 * b_{eff} * (h_d - h_{pb}) = 46334,26 kN

The value for a for use in a contry should lie between 0,8 - 1,0 and me be found in a National Annex.
The Recommended Value for a is 1,0

a_c = 1,00

z_{pl} = $N_{pl,a,Rd}$ / (10 * a_c * f_{cd} * b_{eff}) = 5,45 cm

Þ The plastic neutral axis is in the concrete chord above the profile steel sheeting.

Distance to centre of gravity:

a =	$h / 2 + h_d - z_{pl} / 2$	=	33,27 cm
$\Rightarrow M_{pl,Rd}$ =	$N_{pl,a,Rd} * a / 100$	=	907,30 kNm
$V_{pl,Rd}$ =	$A_v * f_{yk} / (10 * \sqrt{3} * \gamma_a)$	=	666,68 kN

Structural verifications:

$M_{Ed} / M_{pl,Rd}$	=	**1,13 < 1**
$V_{Ed} / V_{pl,Rd}$	=	**0,51 < 1**

Longitudinal shear load capacity and shear connection:

h_b / d = 5,26 > 4 -> shear connectors are ductile

α = IF(h_b /d>4; 1; 0.2 * ((h_b/d)+1)) = 1,00

Marginal force of a shear connector:

$$P_{Rd1} = \frac{0{,}8 * f_u * \pi * d^2}{4 * \gamma_{vs} * 10^3} = 81{,}66 \text{ kN}$$

$$P_{Rd2} = \left(0{,}29 * \alpha * d^2 * \frac{\sqrt{f_{ck} * E_{cm}}}{\gamma_{vs}}\right) * 10^{-3} = 73{,}13 \text{ kN}$$

P_{Rd} = MIN(P_{Rd1} ; P_{Rd2}) = 73,13 kN

single-row:

N_r = 1,00 (one row of shear connectors)

$$k_{t1} = \frac{0{,}7}{\sqrt{N_r}} * \frac{b_0}{h_{pb}} * \left(\frac{h_b * 10^{-1}}{h_{pb}} - 1\right) = 1{,}66$$

k_{t2} = 0,85

k_t = MIN (k_{t1} ; k_{t2}) = 0,85

P_{Rd1} = $k_t * P_{Rd}$ = 62,16 kN

double-row:

N_r = 2,00 (two rows of shear connectors)

$$k_{t1} = \frac{0{,}7}{\sqrt{N_r}} * \frac{b_0}{h_{pb}} * \left(\frac{h_b * 10^{-1}}{h_{pb}} - 1\right) = 1{,}17$$

k_{t2} = 0,70

k_t = MIN (k_{t1} ; k_{t2}) = 0,70

P_{Rd2} = $k_t * P_{Rd}$ = 51,19 kN

Possible shear connector configurations:

7 pairs of shear connectors at the end of the beam Anzp = 7,00

33 individual shear connectors in the span Anze = 33,00

aufn_V = Anze * P_{Rd1} + 2 * Anzp * P_{Rd2} = 2767,94 kN

Structural verification:

$N_{pl,a,Rd}$ / aufn_V = **0,99 < 1**

Structural verification against diagonal strut failure according to NAD for Germany:

A_{cv} = 100 * (h_d - h_{pb}) = 1090,00 N/m

Normal concrete h = 1,00

$n_{Rd,2} = \dfrac{0,2 * A_{cv} * h * f_{ck}}{g_c * 10}$ = 363,33 kN/m

$n_{Ed} = \dfrac{P_{Rd1}}{e_L * 10^{-2}}$ = 414,40 kN/m

$n_{Ed,li}$ = n_{Ed} *1/2 = 207,20 kN/m

$n_{Ed,li}$ / $n_{Rd,2}$ = 0,57 < 1

This calculation disregards the input from profile steel sheeting!

Cross reinforcement necessary:

NAD τ_{Rd} = $0,09 * \sqrt[3]{f_{ck}}$ = 0,26 MN/mm²

$n_{pd} = \dfrac{A_p * f_{yp}}{g_{ap} * 10}$ = 397,60 kN/m

$n_{Rd,3}$ = 2.5 * A_{cv} * h * (t_{Rd} / 10) + n_{pd} = 468,45 kN/m

Structural verification of diagonal strut:

$n_{Ed,li}$ / $n_{Rd,3}$ = 0,44 < 1

Working state analysis of composite beams:

Form factors:

Short term load at point of time t =0:

n_0 = E_a / E_{cm} = 6,89

Effective component thickness:

Beam width in air u = 360,00 cm

h_o = 2 * h_d *u / u = 32,00 cm

Final creep value t_0 =14 days

according to EC2 Appendix1 j = 2,70

Creep under continuous load nj = n_0 * 3 = 20,67

Shrinkage and creep n_S = n_0 * 2 = 13,78

A_c = 100 * b_{eff} * (h_d - h_{pb}) = 3270,00 cm²

I_c = 100 * b_{eff} * (h_d - h_{pb})³ / 12 = 32375,72 cm²m²

$$z'_{s0} = \frac{A \cdot \left(\frac{h}{2} + h_d\right) + \frac{A_c}{n_0} \cdot \left(\frac{h_d - h_{pb}}{2}\right)}{A + \frac{A_c}{n_0}} = 10{,}07 \text{ cm}$$

$$\left(I_y + \frac{I_c}{n_0}\right) \cdot 10^{-4} + \left(A \cdot \left(\frac{h}{2} + h_d\right)^2 + \frac{A_c}{n_0} \cdot \left(\frac{h_d - h_{pb}}{2}\right)^2\right) \cdot 10^{-4} = 15{,}14 \text{ cm}^2\text{m}^2$$

$$\left(z'_{s0} \cdot 10^{-2}\right)^2 \cdot \left(A + \frac{A_c}{n_0}\right) \cdot (-1) = -5{,}67 \text{ cm}^2\text{m}^2$$

I_{i0} = **9,47 cm²m²**

$$z'_{sj} = \frac{A \cdot \left(\frac{h}{2} + h_d\right) + \frac{A_c}{n_j} \cdot (h_d - h_{pb})/2}{A + A_c / n_j} = 16{,}09 \text{ cm}$$

$$\left(I_y + \frac{I_c}{n_j}\right) \cdot 10^{-4} + \left(A \cdot \left(\frac{h}{2} + h_d\right)^2 + \frac{A_c}{n_j} \cdot \left(\frac{h_d - h_{pb}}{2}\right)^2\right) \cdot 10^{-4} = 13{,}89 \text{ cm}^2\text{m}^2$$

$$\left(z's_j \cdot 10^{-2}\right)^2 \cdot \left(A + \frac{A_c}{n_j}\right) \cdot (-1) = -6{,}28 \text{ cm}^2\text{m}^2$$

I_{iphi} = **7,61 cm²m²**

I_{ij} = I_{iphi} = **7,61 cm²m²**

$$z'_{sS} = \frac{A \cdot \left(\frac{h}{2} + h_d\right) + \frac{A_c}{n_S} \cdot (h_d - h_{pb})/2}{A + A_c / n_S} = 13{,}47 \text{ cm}$$

$$\left(I_y + \frac{I_c}{n_S}\right) \cdot 10^{-4} + \left(A \cdot \left(\frac{h}{2} + h_d\right)^2 + \frac{A_c}{n_S} \cdot \left(\frac{h_d - h_{pb}}{2}\right)^2\right) \cdot 10^{-4} = 14{,}20 \text{ cm}^2\text{m}^2$$

$$\left(z'_{sS} \cdot 10^{-2}\right)^2 \cdot \left(A + \frac{A_c}{n_S}\right) \cdot (-1) = -5{,}84 \text{ cm}^2\text{m}^2$$

I_{iS} = **8,36 cm²m²**

Calculation of midspan deflection:

Release of temporary prop at point of time t =0

$B = g_{1k} * 1{,}25 * L/2$ = 108,75 kN

$f_{B0} = \dfrac{10^{2} * B * L^{3}}{48 * E_a * 10^{-1} * I_{i0}}$ = 1,97 cm

Assumption: 40% of the live load as a permanent load at point of time t =0:

Ratio= 0,40

$f_{g2,0} = \dfrac{\frac{5}{384} * 10^{2} * (g_{2k} + Ratio * q_k) * L^{4}}{E_a * 10^{-1} * I_{i0}}$ = 2,03 cm

$\mathbf{f_0 = f_{B0} + f_{g2,0}}$ = **4,00 cm**

from imposed load (short term ratio):

Ratio2 = 1 - Ratio = 0,60

$f_q = \dfrac{\frac{5}{384} * 10^{2} * (Ratio2 * q_k) * L^{4}}{E_a * 10^{-1} * I_{i0}}$ = 1,47 cm

From removal of temporary prop at point of time t =¥:

$f_B = f_{B0} * I_{i0} / I_i j$ = 2,45 cm

From $g_{k2} + q_{permanent}$ at point of time t =¥:

$f_{g2} = f_{g2,0} * I_{i0} / I_i j$ = 2,53 cm

$\mathbf{f_d = f_B + f_{g2}}$ = **4,98 cm**

Creep under continuous load only:

$f_k = f_d - f_0$ = 0,98 cm

From shrinkage at point of time t =¥:

Final shrinkage value e_{cS} = $325*10^{-6}$

$N_{sch} = (A_c * E_a / n_S * e_{cS}) / 10$ = 1619,58 kN

$z_{sch} = z'_{sS} - (h_d - h_{pb})/2$ = 8,02 cm

$M_{sch} = N_{sch} * z_{sch} / 100$ = 129,89 kN

$f_{sch} = \dfrac{\frac{1}{8} * M_{sch} * 10^{2} * L^{2}}{E_a * 10^{-1} * I_{iS}}$ = 1,33 cm

Maximum deflection:

$max\_f = f_B + f_{g2} + f_{sch} + f_q$ = 7,78 cm

Recommended camber:

$f_0 = f_{B0} + f_{g2,0}$ = 4,00 cm

Maximum deflection in final state:

$f = max\_f - f_0$ = 3,78 cm

Structural verification:

$$\frac{f}{100} / \frac{L}{250} = \underline{\mathbf{0{,}79 \leq 1}}$$

The deflection in the working state meets the required standard!

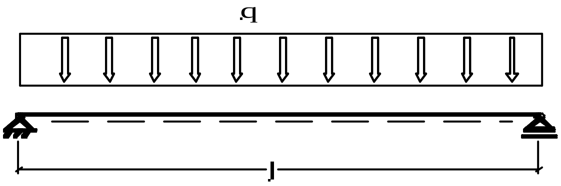
Vibration behaviour:

$$\omega = \frac{10 \cdot \pi^2}{L^2} \cdot \sqrt{\frac{E_a \cdot I_{i0}}{(g_{1k} + g_{2k}) \cdot 10 \cdot 9{,}81}} = 20{,}69\ (1/s)$$

$$f = \frac{\omega}{2 \cdot \pi} = 3{,}29\ \text{Hz}$$

The natural frequency should be no less than 3 Hz.
For sports or dance halls, it should be no less than 5 Hz.

POS.: Composite beams - Ultimate load bearing capacity:



System:

Beam length L =	12,00 m
Number of temporary props a =	1,00
Beam distance s =	3,60 m
Flange width b_0 =	12,60 cm

Materials:

Concrete =	SEL("concrete/EC"; Name;)	=	C25/30
Steel =	SEL("steel/EC"; NameEN;)	=	S355
Beam type =	SEL("steel/profils"; Name;)	=	IPE
Nominal height NH =	SEL("steel/"type; NH;)	=	450
A =	TAB("steel/"type; A; NH=NH)	=	98,80 cm²
h =	TAB("steel/"type; h; NH=NH)/10	=	45,00 cm
t =	TAB("steel/"type; s; NH=NH)/10	=	0,94 cm
I_y =	TAB("steel/"type; I_y ; NH=NH)	=	33740,00 cm4
M_{pl} =	TAB("steel/"type; M_{plyd} ; NH=NH)	=	373,00 kNm
M_{pl} =	IF(Steel="S355";1,5;1)*M_{pl}	=	559,50 kNm

Shear connectors Æ 22

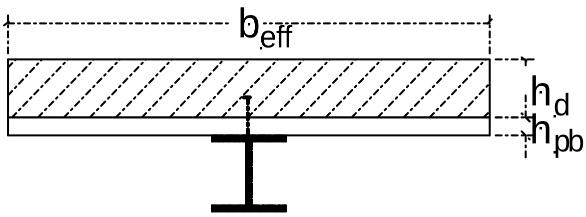
h_b =	100,00 mm
d =	22,00 mm
Distance of shear connectors e_L =	15,00 cm
Ultimate strength f_u =	450,00 N/mm²

Load:

g_{k1} =	15,28 kN/m
g_{k2} =	7,74 kN/m (after removal of temporary props)
q_k =	18,00 kN/m

Partial safety factors:

Dead load γ_G =	1,35
Imposed Load γ_Q =	1,50
Concrete γ_c =	1,50
Construction steel γ_a =	1,10
Profile steel sheeting γ_{ap} =	1,10
Longitudinal shear γ_{vs} =	1,25



Floor thickness h_d =	16,00 cm
Thickness of profile steel sheeting h_{pb} =	5,10 cm
Cross-sectional area of plate A_p =	15,62 kN/m

Material properties:

E_{cm} =	TAB("concrete/EC"; E_{cm}; Name=Concrete)	=	30500,00 N/mm²
f_{ck} =	TAB("concrete/EC"; fck; Name=Concrete)	=	25,00 N/mm²
f_{ctk005} =	TAB("concrete/EC"; fctk05; Name=Concrete)	=	1,80 N/mm²
f_{cd} =	f_{ck} / g_c	=	16,67 N/mm²
E_a =	TAB("steel/EC"; E; NameEN=Steel)	=	210000,00 N/mm²
f_{yk} =	TAB("steel/EC"; fy; NameEN=Steel)	=	355,00 N/mm²
f_{yd} =	f_{yk} / g_a	=	322,73 N/mm²

Holorib sheeting in accordance with building regulations approval:

f_{yp} = 280,00 N/mm²

Conformation of fitness for purpose:

Stress resultants in ultimate state analysis of fitness for purpose:

r_d =	g_G * (g_{k1} + g_{k2}) + g_Q * q_k	=	58,08 kN/m
M_{Ed} =	r_d * L² / 8	=	1045,44 kNm
V_{Ed} =	r_d *L / 2	=	348,48 kN

Cross section load bearing capacity with full shear connection:

b_{eff} =	2 * L / 8	=	3,00 m
b_{eff} =	MIN(b_{eff} ; s)	=	**3,00 m**
A_v =	1.04 * h * t	=	43,99 cm²
$N_{pl,a,Rd}$ =	A * f_{yd} / 10	=	3188,57 kN
N_{cd} =	0.85 * f_{cd} * 10 * b_{eff} * (h_d - h_{pb})	=	4633,43 kN

The value for a for use in a contry should lie between 0,8 - 1,0 and me be found in a National Annex.
The Recommended Value for a is 1,0

a_c = 1,00

z_{pl} = $N_{pl,a,Rd}$ / (10 * a_c * f_{cd} * b_{eff}) = 6,38 cm

Þ The plastic neutral axis is in the concrete chord above the profile steel sheeting.

Distance to centre of gravity:

$a = h / 2 + h_d - z_{pl} / 2 = 35,31$ cm

$\Rightarrow M_{pl,Rd} = N_{pl,a,Rd} * a / 100 = 1125,88$ kNm

$$V_{pl,Rd} = \frac{A_v * f_{yk}}{10 * \sqrt{3} * \gamma_a} = 819,65 \text{ kN}$$

Structural verifications:

$M_{Ed} / M_{pl,Rd}$ = **0,93 < 1**

$V_{Ed} / V_{pl,Rd}$ = **0,43 < 1**

Longitudinal shear capacity and shear connection:

h_b / d = 4,55 > 4 -> shear connectors are ductile

α = IF(h_b /d>4; 1; 0.2 * ((h_b/d)+1)) = 1,00

Marginal force of a shear connector:

$$P_{Rd1} = \frac{0,8 * f_u * \pi * d^2}{4 * \gamma_{vs} * 10^3} = 109,48 \text{ kN}$$

$$P_{Rd2} = \left(0,29 * \alpha * d^2 * \frac{\sqrt{f_{ck} * E_{cm}}}{\gamma_{vs}}\right) * 10^{-3} = 98,05 \text{ kN}$$

P_{Rd} = MIN(P_{Rd1} ; P_{Rd2}) = 98,05 kN

Number of rows of shear connectors N_r = 1,00

$$k_{t1} = \frac{0,7}{\sqrt{N_r}} * \frac{b_0}{h_{pb}} * \left(\frac{h_b * 10^{-1}}{h_{pb}} - 1\right) = 1,66$$

k_{t2} = 0,75

k_t = MIN (k_{t1} ; k_{t2}) = 0,75

$P_{Rd} = k_t * P_{Rd}$ = 73,54 kN

Required number of shear connectors for full shear connection:

req_n = $N_{pl,a,Rd} / P_{Rd}$ + 0.49 = 44

prov_n = 100*L/2/e_L +0.49 = 40

because: req_n / prov_n = 1,10 is a partial connection

The maximum transferrable concrete compression force is:

N_c = prov_n * P_{Rd} = 2941,60 kN

Degree of shear connection η = $N_c / N_{pl,a,Rd}$ = **0,92 < 1**

Reduced plastic bending moment, calculated with linear interpolation:

$M_{Rd} = M_{pl} + \eta * (M_{pl,Rd} - M_{pl})$ = 1080,57 kNm

Structural verification:

M_{Ed} / M_{Rd} = **0,97 < 1**

Note: No reduction in interaction, because maximum moment and maximum lateral shear force do not occur in the same place.

Shear connector configurations permissible?

a) Cross section class 1 or 2 Þ OK

b) req_h = 0.25 + 0.03 * L = 0,61

req_h / h = **0,66 < 1**

c)

$M_{pl,Rd}$ / (2.5 * M_{pl}) = **0,80 < 1**

All conditions a-c have to be met!

Connection of lateral concrete chord:

Lateral shear::

n_{Ed} =	P_{Rd} / (e_L / 100)	=	490,27 kN/m
Relevant:			
$n_{Ed,li}$ =	n_{Ed} * 1 / 2	=	245,13 kN/m
A_{cv} =	(h_d - h_{pb}) * 100	=	1090,00 cm²/m
Normal concrete h =			1,00
$n_{Rd,2}$ =	0.2 * A_{cv} * h * f_{ck} / (g_c *10)	=	363,33 kN/m

Structural verification against diagonal strut failure:

$n_{Ed,li}$ / $n_{Rd,2}$ = **0,67 < 1**

This calculation disregards the input from profile steel sheeting!

EC4 t_{Rd1} =	0.25 * f_{ctk005} / g_c	=	0,30 MN/mm²
NAD t_{Rd2} =	0,09 * ^{3}Ö f_{ck}	=	0,26 MN/mm²
t_{Rd} =	MIN (t_{Rd1} ; t_{Rd2})	=	0,26 MN/mm²
n_{pd} =	A_p * f_{yp} /(g_{ap} * 10)	=	397,60 kN/m
$n_{Rd,3}$ =	2.5 * A_{cv} * h * (t_{Rd} / 10) + n_{pd}	=	468,45 kN/m

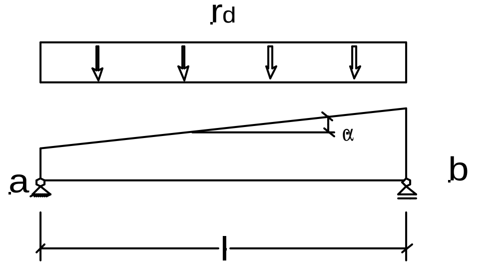
Structural verification of diagonal strut:

$n_{Ed,li}$ / $n_{Rd,3}$ = **0,52 < 1**

The cross reinforcement has been ignored in this calculation.

In the construction, provision should be made for 0.2% or the relevant cross section area A_{cv} that may also be taken into account as part of the bending reinforcement to absorb the negative moment (moment at support) of the transversal continuous composite floor.

Half-span roof:



System:

Beam length l =	7,00 m
Beam width b =	14,00 cm
Beam height an a h_a =	26,00 cm
Pitch α =	6,00 °
Support length t =	10,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS14h
$f_{m,g,k}$ =	TAB("wood/EC"; fm.k; SG=SG)	=	28,00 N/mm²
$f_{v,g,k}$ =	TAB("wood/EC"; fv.k; SG=SG)	=	2,70 N/mm²
$f_{t,90,g,k}$ =	TAB("wood/EC"; ft,90.k; SG=SG)	=	0,45 N/mm²
$f_{c,90,k}$ =	TAB("wood/EC"; fc,90.k; SG=SG)	=	5,50 N/mm²
$E_{0,mean}$ =	TAB("wood/EC"; E0.mean; SG=SG)	=	12500,00 N/mm²
γ_M =	1,30		
γ_S =	1,10		

Load:

r_d = 12,50 kN/m

Calculation:

$$f_{m,g,d} = f_{m,g,k} * \frac{k_{mod}}{g_M} = 19{,}38 \text{ N/mm}^2$$

$$f_{v,g,d} = f_{v,g,k} * \frac{k_{mod}}{g_M} = 1{,}87 \text{ N/mm}^2$$

$$f_{c,90,d} = f_{c,90,k} * \frac{k_{mod}}{g_M} = 3{,}81 \text{ N/mm}^2$$

$$f_{t,90,g,d} = f_{t,90,g,k} * \frac{k_{mod}}{g_M} = 0{,}31 \text{ N/mm}^2$$

Stress resultants for analysis of load bearing capacity:

$$M_d = r_d * \frac{l^2}{8} = 76{,}56 \text{ kNm}$$

$$V_d = \frac{l}{2} * r_d = 43{,}75 \text{ kN}$$

$h_{s,req} = 150 * \dfrac{V_d}{b * f_{v,g,d}}$ = 250,67 mm

$\dfrac{h_{s,req}}{10 * h_a}$ = **0,96 < 1**

$h_b = h_a + l * 100 * TAN(\alpha)$ = 99,57 cm

Point of maximum stress:

$x = \dfrac{l}{1 + \dfrac{h_b}{h_a}}$ = 1,45 m

$M_{x,d} = V_d * x - r_d * x^2 * 0,5$ = 50,30 kNm

$W_{x,req} = 1100 * \dfrac{M_{x,d}}{f_{m,g,d}}$ = 2855,01 cm³

$h_x = h_a + x * 100 * TAN(\alpha)$ = 41,24 cm

$W_x = b * \dfrac{h_x^2}{6}$ = 3968,39 cm³

$\dfrac{W_{x,req}}{W_x}$ = **0,72 < 1**

Required area moment of second degree at a deflection of l/300:

$I_{req} = 3130000 * \dfrac{M_d}{1,4} * \dfrac{l}{E_{0,mean}}$ = 95853,12 cm⁴

$h_{m,req} = 0,1 * \sqrt[3]{12000 * \dfrac{I_{req}}{b}}$ = 43,47 cm

$h_m = \dfrac{h_b - h_a}{2} + h_a$ = 62,78 cm

$\dfrac{h_{m,req}}{h_m}$ = **0,69 < 1**

Analysis of load-bearing capacity:
Shearing stress at footing a:

$\tau_{a,d} = 15 * \dfrac{V_d}{b * h_a}$ = 1,80 N/cm²

$\dfrac{t_{a,d}}{f_{v,g,d}}$ = **0,96 < 1**

Stresses in the edge parallel to the grain:

$\sigma_{m,0,d} = 0,001 * (1 + 4 * (TAN(\alpha))^2) * 6 * 10^6 * \dfrac{M_{x,d}}{b * h_x^2}$ = 13,24 N/mm²

$\dfrac{s_{m,0,d}}{f_{m,g,d}}$ = **0,68 < 1**

Stresses in the edge with grains at an angle:

$\sigma_{m,a,d}$ = $0{,}001 * (1 - 4 * (TAN(\alpha))^2) * 6 * 10^6 * \dfrac{M_{x,d}}{b * h_x^{\,2}}$ = 12,12 N/mm²

$f_{m,\alpha,d} = \dfrac{f_{m,g,d}}{\dfrac{f_{m,g,d}}{f_{c,90,d}} * \sin(a)^2 + \cos(a)^2}$ = 18,55 N/mm²

$\sigma_{m,\alpha,d}$ / $f_{m,\alpha,d}$ = **0,65 < 1**

Bearing pressure:
Assumption: l < 150 mm;l_1 > 150 mm ; $k_{c,90}$ according to Tab. 5.1.5

$\Rightarrow k_{c,90}$ = 1,0

req_A = $10 * \dfrac{V_d}{f_{c,90,d} * k_{c,90}}$ = 114,83 cm²

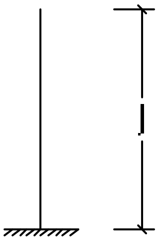
req_t = $\dfrac{req\_A}{b}$ = 8,20 cm

$\dfrac{req\_t}{t}$ = **0,82 < 1**

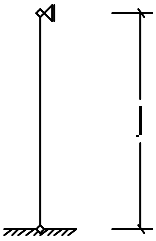
Buckling lengths:

Euler's crippling load:

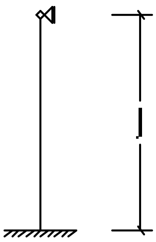
l = 5,00 m



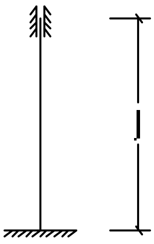
l_{ef} = 2*l = 10,00 m



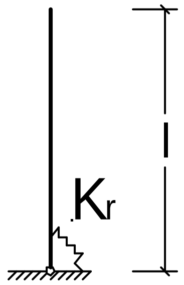
l_{ef} = l = 5,00 m



l_{ef} = 0,7*l = 3,50 m



l_{ef} = 0,5*l = 2,50 m



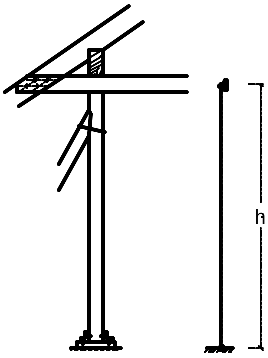
Sorting grade SG = SEL("Wood/EC"; SG;) = S10

$E_{0,mean}$ = TAB("Wood/EC"; $E_{0.mean}$; SG=SG) = 11000,00 N/mm²

K_r = 100000,00 1/N

l_{ef} = $l * \sqrt{4 + \frac{\pi^2 * E_{0,mean}}{l * K_r}}$ = 10,27 m

Roof post:



System:

Post height h =	3,40 m
Cross-sectional width b =	14,00 cm
Cross-sectional thickness d =	14,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90

Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
$f_{c,0,k}$ =	TAB("wood/EC"; fc,0,k; SG=SG)	=	21,00 N/mm²
$f_{c,90,k}$ =	TAB("wood/EC"; fc,90,k; SG=SG)	=	5,00 N/mm²
γ_M = 1,30			

Load at the head:

$V_{c,d}$ = 50,40 kN

Calculation:

Buckling length l_{ef} =	h	=	3,40 m
A =	b *d	=	196,00 cm²
i =	$\frac{1}{\text{Ö}12}$*MIN(b;d)	=	4,04 cm
Degree of slenderness λ_{min} =	$100 * \frac{l_{ef}}{i}$	=	84,16 > 30

⇒ Buckling proof required!

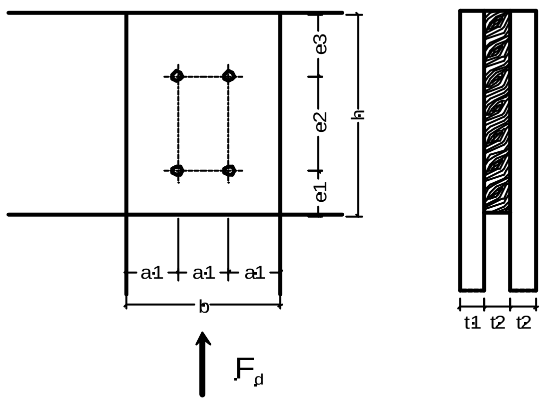
k_c =	TAB("wood/ECbuckl"; kc; SG=SG; l=l$_{min}$)	=	0,427
$s_{c,0,d}$ =	$10 * \frac{V_{c,d}}{A}$	=	2,57 N/mm²
$f_{c,0,d}$ =	$f_{c,0,k} * \frac{k_{mod}}{g_M}$	=	14,54 N/mm²
$f_{c,90,d}$ =	$f_{c,90,k} * \frac{k_{mod}}{g_M}$	=	3,46 N/mm²

Structural verifications:

Buckling: $\frac{s_{c,0,d}}{k_c * f_{c,0,d}}$ = **0,41** £**1**

Lateral pressure: $\frac{s_{c,0,d}}{f_{c,90,d}}$ = **0,74** £**1**

Beam connection:



System:

Distance a_1 =			4,00 cm
Width b =	3*a_1	=	12,00 cm
Distance e_1 =			4,00 cm
Distance e_2 =			8,50 cm
Distance e_3 =			5,50 cm
Thickness t_1=			6,00 cm
Thickness t_2 =			8,00 cm

Load:

F_d = 40,00 kN

Material:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} = TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)		=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
γ_M =	1,30		

Dowel: S235

Dowel diameter d =	SEL("wood/DPin"; d;)/10	=	1,20 cm
$f_{u,k}$ =	360,00 N/mm²		
γ_S =	1,10 N/mm²		

Calculation:

$$M_{y,d} = 0{,}8 * f_{u,k} * \frac{d^3}{6 * g_S} * 10^3 = 75403{,}6 \text{ Nmm}$$

Post:

$$f_{h,1,d} = 0{,}082 * \rho_k * (1\text{-}0{,}1\text{*}d) * \frac{k_{mod}}{g_M} = 16{,}87 \text{ N/mm}^2$$

Support:

$$k_{90} = 1.35 + 0.15 * d \qquad = \quad 1,53$$

$$f_{h,2,d} = 0,082 * \rho_k * (1\text{-}0,1*d) * \frac{k_{mod}}{g_M * k_{90}} \qquad = \quad 11,03 \text{ N/mm}^2$$

$$\beta = \frac{f_{h,2,d}}{f_{h,1,d}} \qquad = \quad 0,654$$

$$R_{D1} = f_{h,1,d} * t_1 * d * 100 \qquad = \quad 12146 \text{ N}$$

$$R_{D2} = 0,5 * f_{h,1,d} * t_2 * d * \beta * 100 \qquad = \quad 5296 \text{ N}$$

$$R_{D3} = 1,1 * f_{h,1,d} * 10^2 * t_1 * \frac{d}{2+b} * \left(\sqrt{2 * b * (1+b) + \frac{4 * b * (2+b) * M_{y,d}}{f_{h,1,d} * 10^3 * d * t_1^2}} - b \right) \qquad = \quad 5254 \text{ N}$$

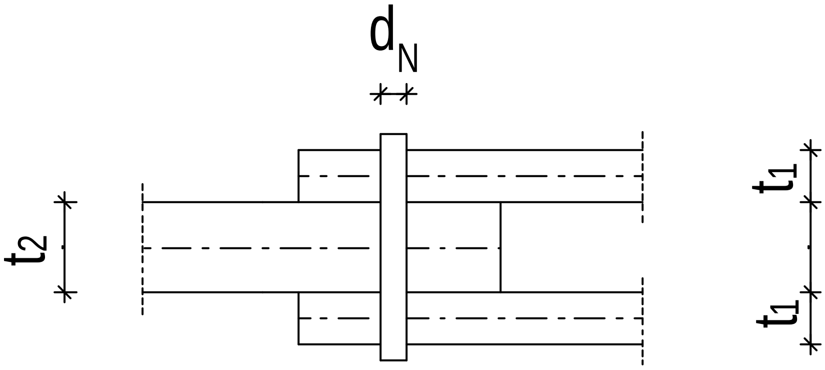
$$R_{D4} = 1,1 * \sqrt{\frac{2 * b}{1+b}} * \sqrt{2 * M_{y,d} * f_{h,1,d} * d * 10} \qquad = \quad 5405 \text{ N}$$

$$R_D = 0,001 * \text{MIN}(R_{D1};R_{D2};R_{D3};R_{D4}) \qquad = \quad \underline{\mathbf{5,25\ kN}}$$

$$n = \frac{F_d}{2 * R_D} \qquad = \quad \underline{\mathbf{3,8}}$$

selected: 4 PIN ∅ 12mm S 235 l = 20 cm

Tensile splice Design of strut



System:

Timber thickness t_1 = 8,00 cm

Timber thickness t_2 = 12,00 cm

Load:

F_d = 116,00 kN

Materials: NH S 10

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³

γ_M = 1,30

γ_S = 1,10

Dowel: DPin Æ 16mm S 275

Dowel diameter d = SEL("wood/DPin"; d;)/10 = 0,80 cm

$f_{u,k}$ = 360,00 N/mm²

Calculation:

Minimum distances according to Tab.6.6a:

a_1 =	7* d	=	5,60 cm
a_2 =	3 * d	=	2,40 cm
$a_{3,t}$ =	7* d	=	5,60 cm
$a_{4,t}$ =	3 * d	=	2,40 cm

Design value of bearing stress resistance:

$$f_{h,0,d} = 0{,}082 * \rho_k * (1-0{,}1*d) * \frac{k_{mod}}{g_M} = 17{,}64 \text{ N/mm}^2$$

$$M_{y,d} = 1000 * 0{,}8 * \frac{f_{u,k} * d^3}{6 * g_S} = 22341{,}8 \text{ Nmm}$$

$$k_M = 10 * \frac{t_1}{\ddot{O}\frac{M_{y,d}}{f_{h,0,d} * d * 10}} = 6{,}36$$

R_{D1} = $100 * f_{h,0,d} * t_1 * d$ = 11289,60 N

R_{D2} = $100 * 0{,}5 * f_{h,0,d} * t_2 * d$ = 8467,20 N

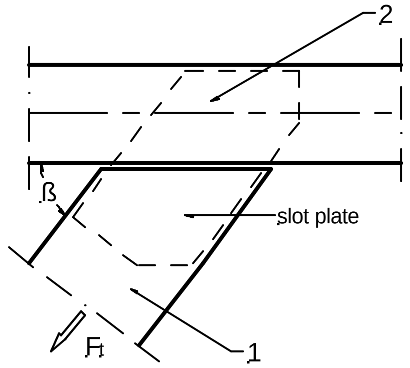
R_{D3} = $100 * 0{,}367 * f_{h,0,d} * t_1 * d * \left(2 * \sqrt{1 + \frac{3}{k_M^2}} - 1\right)$ = 4445,08 N

R_{D4} = $155{,}6 * f_{h,0,d} * t_1 * \frac{d}{k_M}$ = 2762,05 N

R_D = $0{,}001 * \mathrm{MIN}(R_{D1};R_{D2};R_{D3};R_{D4})$ = **2,76 kN**

required number of dowels $n = \frac{F_d}{2 * R_D}$ = **21,0**

Tension diagonal:



System:

Rod 1 : **1 * 160mm * 160mm Glulam**

Rod 2 : **1 * 160mm * 180mm Glulam**

System angle β =	60,0 °
Timber thickness t_1 =	16,00 cm
Timber thickness t_2 =	16,00 cm
Thickness of slotted plate t_3 =	0,80 cm

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	2

⇒ k_{mod} = TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC) = 0,80

Strength grade SG=	SEL("wood/EC"; SG;)	=	BS11
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	410,00 kg/m³

γ_M = 1,30

γ_S = 1,10

Dowel: DPin Æ 16mm S 275

Dowel diameter d =	SEL("wood/DPin"; d;)/10	=	1,60 cm

$f_{u,k}$ = 430,00 N/mm²

$F_{t,d}$ = 80,79 kN

Calculation:

Minimum distances of Rod1 according to Tab.6.6a:

a_1 =	7* d	=	11,20 cm
a_2 =	3 * d	=	4,80 cm
$a_{3,t}$ =	7* d	=	11,20 cm
$a_{4,t}$ =	3 * d	=	4,80 cm

Minimum distances Rod2 according to Tab.6.6a:

a_1 =	(3 + 4 * ABS(COS(β)))* d	=	8,00 cm
a_2 =	3 * d	=	4,80 cm
$a_{3,t}$ continuous top chord			
$a_{4,t}$ =	(2 + 2 * ABS(SIN(β))) * d	=	5,97 cm

Rod 1 :

$f_{h,1,d} = \quad 0,082 * \rho_k * (1-0,1*d) * \dfrac{k_{mod}}{g_M} \quad = \quad 17,38\ \text{N/mm}^2$

Rod 2 :

$f_{h,0,d} = \quad 0,082 * \rho_k * (1-0,1*d) * \dfrac{k_{mod}}{g_M} \quad = \quad 17,38\ \text{N/mm}^2$

$k_{90} = \quad 1,35 + 0,15 * d \quad = \quad 1,59$

$f_{h,2,d} = \quad \dfrac{f_{h,0,d}}{k_{90} * \sin(b)^2 + \cos(b)^2} \quad = \quad 12,05\ \text{N/mm}^2$

$M_{y,d} = \quad 10^3 * 0,8 * f_{u,k} * \dfrac{d^3}{6 * g_S} \quad = \quad 213488,5\ \text{Nmm}$

Design value of drift pins in Rod 1:

$R_{D1} = \quad 100 * f_{h,1,d} * \left(\dfrac{t_1}{2} - t_3\right) * d \quad = \quad 20022\ \text{N}$

$R_{D2} = \quad 110 * f_{h,1,d} * \left(\dfrac{t_1}{2} - t_3\right) * d * \left(\sqrt{2 + \dfrac{4 * M_{y,d}}{f_{h,1,d} * 10^3 * d * \left(\dfrac{t_1}{2} - t_3\right)^2}} - 1\right) \quad = \quad 13437\ \text{N}$

$R_{D3} = \quad 1,5 * \sqrt{20 * M_{y,d} * f_{h,1,d} * d} \quad = \quad 16345\ \text{N}$

$R_D = \quad 0,001 * \text{MIN}(R_{D1};R_{D2};R_{D3}) \quad = \quad$ **13,44 kN**

$n = \quad \dfrac{F_{t,d}}{2 * R_D} \quad = \quad$ **3,0**

selected: 4 DPin ∅ 16 S 275 l = 16cm

Design value of drift pins in Rod 2:

$R_{D1} = \quad 100 * f_{h,2,d} * \left(\dfrac{t_1}{2} - t_3\right) * d \quad = \quad 13882\ \text{N}$

$R_{D2} = \quad 110 * f_{h,2,d} * \left(\dfrac{t_1}{2} - t_3\right) * d * \left(\sqrt{2 + \dfrac{4 * M_{y,d}}{f_{h,2,d} * 10^3 * d * \left(\dfrac{t_1}{2} - t_3\right)^2}} - 1\right) \quad = \quad 10528\ \text{N}$

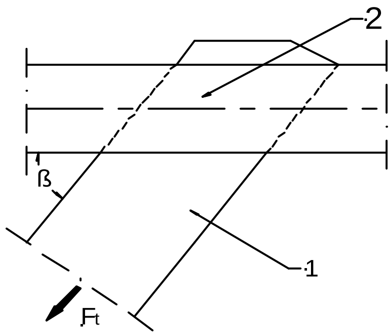
$R_{D3} = \quad 1,5 * \sqrt{20 * M_{y,d} * f_{h,2,d} * d} \quad = \quad 13610\ \text{N}$

$R_D = \quad 0,001 * \text{MIN}(R_{D1};R_{D2};R_{D3}) \quad = \quad$ **10,53 kN**

$n = \quad \dfrac{F_{t,d}}{2 * R_D} \quad = \quad$ **3,8**

selected: 4 DPin ∅ 16 S 275 l = 16cm

Tension diagonal:



System:

Rod 1 : 2 * 60mm * 140mm
Rod 2 : 1 * 100mm * 180mm

System angle β = 60,00 °
Timber thickness t_1 = 6,00 cm
Timber thickness t_2 = 10,00 cm

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80

Strength grade SG=	SEL("wood/EC"; SG;)	=	S7
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	350,00 kg/m³

γ_M = 1,30
γ_S = 1,10

Dowel: DPin Æ 12mm

Dowel diameter d =	SEL("wood/DPin"; d;)/10	=	1,20 cm
$f_{u,k}$ =	360,00 N/mm²		

Load:

$F_{t,d}$ = 40,00 kN

Calculation:

Minimum distances of Rod1 according to Tab.6.6a:

a_1 =	7* d	=	8,40 cm
a_2 =	3 * d	=	3,60 cm
$a_{3,t}$ =	7* d	=	8,40 cm
$a_{4,t}$ =	3 * d	=	3,60 cm

Minimum distances Rod2 according to Tab.6.6a:

a_1 =	(3 + 4 * ABS(COS(β)))* d	=	6,00 cm
a_2 =	3 * d	=	3,60 cm
$a_{3,t}$ continuous top chord			
$a_{4,t}$ =	(2 + 2 * ABS(SIN(β))) * d	=	4,48 cm

Rod 1 :

$f_{h,1,d} = 0{,}082 * \rho_k * (1-0{,}1*d) * \dfrac{k_{mod}}{\gamma_M}$ = 15,54 N/mm²

Rod 2 :

$f_{h,0,d} = 0{,}082 * \rho_k * (1-0{,}1*d) * \dfrac{k_{mod}}{\gamma_M}$ = 15,54 N/mm²

$k_{90} = 1{,}35 + 0{,}15 * d$ = 1,53

$f_{h,2,d} = \dfrac{f_{h,0,d}}{k_{90} * \sin(\beta)^2 + \cos(\beta)^2}$ = 11,12 N/mm²

$\beta = \dfrac{f_{h,2,d}}{f_{h,1,d}}$ = 0,716

$M_{y,d} = 10^3 * 0{,}8 * f_{u,k} * \dfrac{d^3}{6 * \gamma_S}$ = 75403,6 Nmm

$R_{D1} = f_{h,1,d} * t_1 * d * 100$ = 11189 N

$R_{D2} = 0{,}5 * f_{h,1,d} * t_2 * d * \beta * 100$ = 6676 N

$R_{D3} = 1{,}1 * f_{h,1,d} * 10^2 * t_1 * \dfrac{d}{2+\beta} * \left(\sqrt{2 * \beta * (1+\beta) + \dfrac{4 * \beta * (2+\beta) * M_{y,d}}{f_{h,1,d} * 10^3 * d * t_1^2}} - \beta\right)$ = 5026 N

$R_{D4} = 1{,}1 * \sqrt{\dfrac{2 * \beta}{1+\beta}} * \sqrt{2 * M_{y,d} * f_{h,1,d} * d * 10}$ = 5329 N

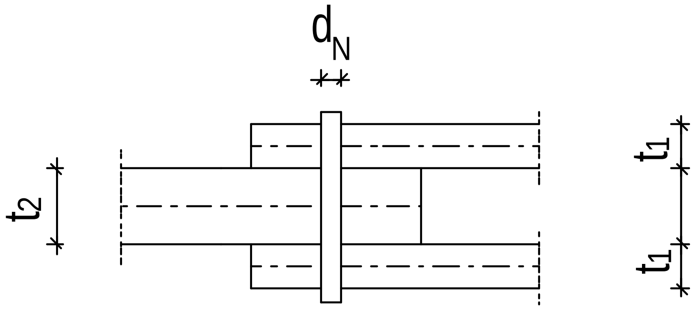
$R_D = 0{,}001 * \text{MIN}(R_{D1};R_{D2};R_{D3};R_{D4})$ = **5,03 kN**

$n = \dfrac{F_{t,d}}{2 * R_D}$ = **4,0**

selected: 4 DPin ∅ 12 S 235 l = 22cm

Nail calculation for each shear joint, thin outside plate:

Thin outside plate



System:

Sheet thickness t_1 = 0,15 cm

Timber thickness t_2 = 4,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	S13
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³

γ_M = 1,30

γ_S = 1,10

Nails: 38x100

Nail length l_N = 10,00 cm

Nail diameter d_N = 0,38 cm

t_1 / d_N = 0,39 < 0,5

otherwise a different calculation applies

Calculation:

pre-drilled:

$f_{h,k}$ = 0.082 * ρ_k * (1-0.1*d_N) = 29,98 N/mm²

not pre-drilled::

$f_{h,k}$ = 0.082 * ρ_k * $(10*d_N)^{-0.3}$ = 20,88 N/mm²

Characteristic value for yield moment of square nails

$$M_{y,d} = 270 * \frac{(10 * d_N)^{2,6}}{g_S} = 7896,02 \text{ Nmm}$$

Characteristic value for yield moment of round nails

$$M_{y,d} = 180 * \frac{(10 * d_N)^{2,6}}{g_S} = 5264,02 \text{ Nmm}$$

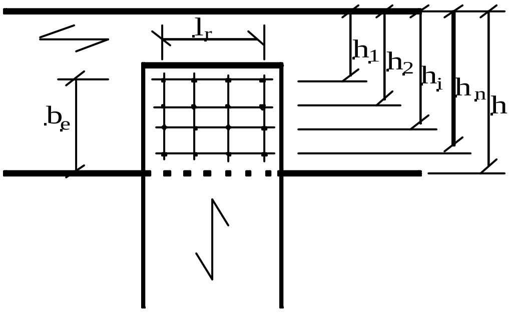
$$f_{h,d} = \frac{k_{mod}}{g_M} * f_{h,k} = 14,46 \text{ N/mm²}$$

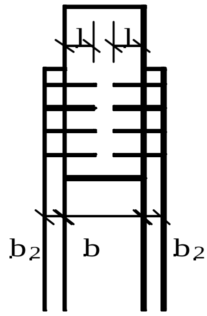
R_{D1} = 100*0,5* $f_{h,d}$ * t_2 * d_N = 1098,96 N

R_{D2} = 1,1 * Ö $\overline{20 * M_{y,d} * f_{h,d} * d_N}$ = 836,65 N

R_D = 0.001 * MIN(R_{D1};R_{D2}) = <u>0,84 kN</u>

Design of a shear connection:





System:

Thickness of outer wood d_2 =	4,00 cm
Nail distance l_r =	6,00 cm
Nail distance b_e =	16,00 cm
Cross beam width b =	16,00 cm
Cross beam height h =	40,00 cm
Height h_1=	24,00 cm
Height h_2 =	28,00 cm
Height h_3 =	32,00 cm
Height h_4 =	36,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS11
Shear and torsion $f_{v,g,k}$ =	TAB("wood/EC"; fv,k; SG=SG)	=	2,70 N/mm²
Rectangular shear $f_{t,90,g,k}$ =	TAB("wood/EC"; ft,90.k; SG=SG)	=	0,45 N/mm²
γ_{MH} =	1,30		
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
Rectangular shear $f_{t,0,k}$ =	TAB("wood/EC"; ft,0,k; SG=SG)	=	14,00 N/mm²

Nails 38x100:

Nail diameter d =			0,38 cm
Nail length l_N =			10,00 cm
Nail strength R_D =	6000*d²	=	866,40 N
γ_F =			1,50

$f_{v,g,d}$ = $f_{v,g,k} * \frac{k_{mod}}{g_{MH}}$ = 1,869 N/mm²

$f_{t,90,g,d}$ = $f_{t,90,g,k} * \frac{k_{mod}}{g_{MH}}$ = 0,312 N/mm²

$f_{t,0,d}$ = $f_{t,0,k} * \frac{k_{mod}}{g_{MH}}$ = 9,692 N/mm²

l_{eff} = 10*MIN(d_2;12*d;l_N-d_2) = 40,00 mm

Calculation of maximum design value:

from nail analysis $F_{90,d,N}$ =	$0.002 * 16 * R_D$	=	27,72 kN
from tension bar $F_{90,d,Z}$ =	$0.2 / \gamma_F * d_2 * l_N * f_{t,0,d}$	=	51,69 kN

detailed structural verification:

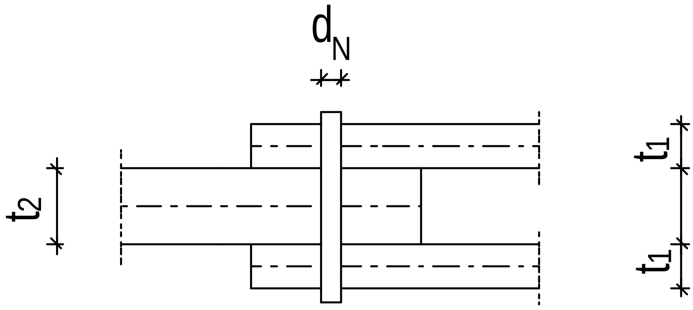
$$\frac{b_e}{h} = 0{,}40 < 0{,}5$$

⇒ detailed structural verification required

b_{ef} =	$2 * l_{eff}$	=	80,00 mm
c =	$\frac{4}{3} * \sqrt{\frac{b_e}{h} * \left(1 - \frac{b_e}{h}\right)^3}$	=	0,392
$l_{r,ef}$ =	$10 * \sqrt{l_r^2 + (c * h)^2}$	=	167,89 mm
A_{ef} =	$l_{r,ef} * b_{ef}$	=	13431,20 mm²
k_r =	$0{,}25 * \left(1 + \left(\frac{h_1}{h_2}\right)^2 + \left(\frac{h_1}{h_3}\right)^2 + \left(\frac{h_1}{h_4}\right)^2\right)$	=	0,685
η =	$1 - 3 * \left(\frac{b_e}{h}\right)^2 + 2 * \left(\frac{b_e}{h}\right)^3$	=	0,648
$F_{90,d}$ =	$\frac{0{,}001}{h * k_r} * 13 * A_{ef}^{0{,}8} * f_{t,90,g,d}$	=	18,34 kN
Decisive $F_{90,d,max}$ =	$MIN(F_{90,d};F_{90,d,N};F_{90,d,Z})$	=	**18,34 kN**

Nail calculation for each shear joint for two different kinds of wood:

two unequal kinds of wood



System:

Timber thickness t_1 = 4,00 cm

Timber thickness t_2 = 4,00 cm

$t_1 \leq t_2$

Materials:

Construction material CM= SEL("wood/kmod"; CM;) = solid wood

Class of load duration CLD= SEL("wood/kmod"; CLD;) = short term

Utility class UC= SEL("wood/kmod"; UC;) = 1

$\Rightarrow k_{mod}$ = TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC) = 0,90

Strength grade SG1= SEL("wood/EC"; SG;) = S13

ρ_{k1} = TAB("wood/EC"; r k; SG=SG1) = 380,00 kg/m³

Strength grade SG2= SEL("wood/EC"; SG;) = MS13

ρ_{k2} = TAB("wood/EC"; r k; SG=SG2) = 400,00 kg/m³

γ_M = 1,30

γ_S = 1,10

Nails: 38x100

Nail length l_N = 10,00 cm

Nail diameter d_N = 0,38 cm

Calculation:

pre-drilled:

$f_{h,k1} = 0.082 * \rho_{k1} * (1-0.1*d_N)$ = 29,98 N/mm²

not pre-drilled::

$f_{h,k1} = 0.082 * \rho_{k1} * (10*d_N)^{-0.3}$ = 20,88 N/mm²

pre-drilled:

$f_{h,k2} = 0.082 * \rho_{k2} * (1-0.1*d_N)$ = 31,55 N/mm²

not pre-drilled::

$f_{h,k2} = 0.082 * \rho_{k2} * (10*d_N)^{-0.3}$ = 21,98 N/mm²

Characteristic value for yield moment of square nails

$$M_{y,d} = 270 * \frac{(10 * d_N)^{2,6}}{g_S} = 7896{,}02 \text{ Nmm}$$

Characteristic value for yield moment of round nails

$$M_{y,d} = 180 * \frac{(10 * d_N)^{2,6}}{g_S} = 5264{,}02 \text{ Nmm}$$

$$f_{h,d1} = \frac{k_{mod}}{g_M} * f_{h,k1} = 14{,}46 \text{ N/mm}^2$$

$f_{h,d2} = \dfrac{k_{mod}}{\gamma_M} * f_{h,k2}$ = 15,22 N/mm²

$\beta = \dfrac{f_{h,d1}}{f_{h,d2}}$ = 0,95

$R_{D1} = 100 * f_{h,d1} * t_1 * d_N$ = 2198 N

$R_{D2} = 100 * 0,5 * f_{h,d1} * t_2 * d_N * \beta$ = 1044 kN

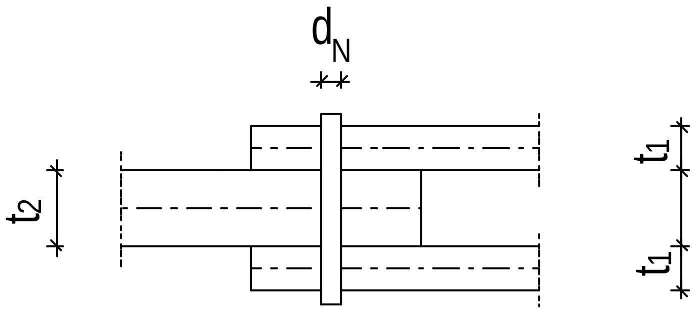
$R_{D3} = 110 * f_{h,d1} * t_1 * \dfrac{d_N}{2+\beta} * \left(\sqrt{2 * \beta * (1+\beta) + \dfrac{4 * \beta * (2+\beta) * M_{y,d}}{f_{h,d1} * 10^3 * d_N * t_1^2}} - \beta \right)$ = 936 N

$R_{D4} = 1,1 * \sqrt{\dfrac{2 * \beta}{1+\beta}} * \sqrt{2 * M_{y,d} * f_{h,d1} * d_N * 10}$ = 826 N

$R_D = 0,001 * MIN(R_{D1};R_{D2};R_{D3};R_{D4})$ = **0,83 kN**

Nail calculation for each shear joint:

Thick outside plate



System:

Sheet thickness t_1 = 0,50 cm

Timber thickness t_2 = 4,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1

$\Rightarrow k_{mod}$ = TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC) = 0,90

Strength grade SG=	SEL("wood/EC"; SG;)	=	S13
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³

γ_M = 1,30

γ_S = 1,10

Nails: 38x100

Nail length l_N = 10,00 cm

Nail diameter d_N = 0,38 cm

otherwise a different calculation applies

Calculation:

pre-drilled:

$f_{h,k}$ = 0.082 * ρ_k * (1-0.1*d_N) = 29,98 N/mm²

not pre-drilled::

$f_{h,k}$ = 0.082 * ρ_k * $(10*d_N)^{-0.3}$ = 20,88 N/mm²

Characteristic value for yield moment of square nails

$$M_{y,d} = 270 * \frac{(10 * d_N)^{2,6}}{g_S} = 7896{,}02 \text{ Nmm}$$

Characteristic value for yield moment of round nails

$$M_{y,d} = 180 * \frac{(10 * d_N)^{2,6}}{g_S} = 5264{,}02 \text{ Nmm}$$

$$f_{h,d} = \frac{k_{mod}}{g_M} * f_{h,k} = 14{,}46 \text{ N/mm²}$$

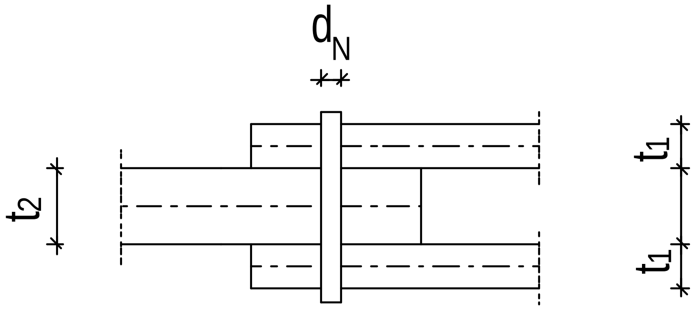
R_{D1} = 100*0,5* $f_{h,d}$ * t_2 * d_N = 1098,96 N

R_{D2} = $1,5 * \text{Ö}\overline{20 * M_{y,d} * f_{h,d} * d_N}$ = 1140,88 N

R_D = 0.001 * MIN(R_{D1};R_{D2}) = <u>1,10 kN</u>

Nail calculation for each shear joint, plate inside:

Plate inside



System:

Timber thickness t_1 = 4,00 cm

Sheet thickness t_2 = 1,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1

$\Rightarrow k_{mod}$ = TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC) = 0,90

Strength grade SG=	SEL("wood/EC"; SG;)	=	S13
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³

γ_M = 1,30

γ_S = 1,10

Nails: 38x100

Nail length l_N = 10,00 cm

Nail diameter d_N = 0,38 cm

<u>Calculation:</u>

pre-drilled:

$f_{h,k}$ = 0.082 * ρ_k * (1-0.1*d_N) = 29,98 N/mm²

not pre-drilled::

$f_{h,k}$ = 0.082 * ρ_k * $(10*d_N)^{-0.3}$ = 20,88 N/mm²

Characteristic value for yield moment of square nails

$$M_{y,d} = 270 * \frac{(10 * d_N)^{2,6}}{g_S} = 7896{,}02 \text{ Nmm}$$

Characteristic value for yield moment of round nails

$$M_{y,d} = 180 * \frac{(10 * d_N)^{2,6}}{\gamma_S} = 5264,02 \text{ Nmm}$$

$$f_{h,d} = \frac{k_{mod}}{\gamma_M} * f_{h,k} = 14,46 \text{ N/mm}^2$$

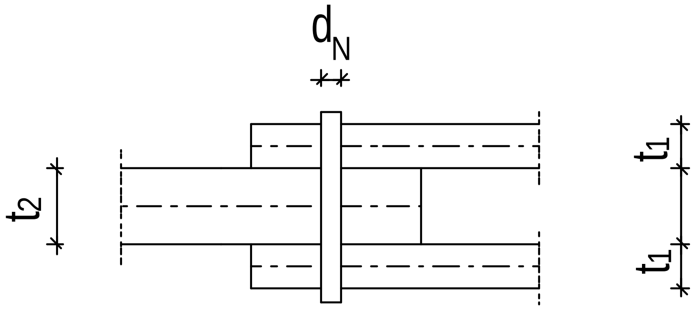
$$R_{D1} = 100 * f_{h,d} * t_1 * d_N = 2198 \text{ N}$$

$$R_{D2} = 110 * f_{h,d} * t_1 * d_N * \left(\sqrt{2 + \frac{4 * M_{y,d}}{f_{h,d} * 10^3 * d_N * t_1^2}} - 1\right) = 1200 \text{ N}$$

$$R_{D3} = 1,5 * \sqrt{20 * M_{y,d} * f_{h,d} * d_N} = 1141 \text{ N}$$

$$R_D = 0.001 * \mathrm{MIN}(R_{D1};R_{D2};R_{D3}) = \underline{\mathbf{1,14\ kN}}$$

Nail calculation for each shear joint:



System:

Timber thickness t_1 = 4,00 cm

Timber thickness t_2 = 4,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood		
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term		
Utility class UC=	SEL("wood/kmod"; UC;)	=	1		
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)			=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)			=	S13
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³		

γ_M = 1,30

γ_S = 1,10

Nails: 38x100

Nail length l_N = 10,00 cm

Nail diameter d_N = 0,38 cm

<u>Calculation:</u>

pre-drilled:

$f_{h,k}$ = 0.082 * ρ_k * (1-0.1*d_N) = 29,98 N/mm²

not pre-drilled::

$f_{h,k}$ = 0.082 * ρ_k * $(10*d_N)^{-0.3}$ = 20,88 N/mm²

Characteristic value for yield moment of square nails

$$M_{y,d} = 270 * \frac{(10 * d_N)^{2,6}}{g_S} = 7896,02 \text{ Nmm}$$

Characteristic value for yield moment of round nails

$$M_{y,d} = 180 * \frac{(10 * d_N)^{2,6}}{\gamma_S} = 5264,02 \text{ Nmm}$$

$$f_{h,d} = \frac{k_{mod}}{\gamma_M} * f_{h,k} = 14,46 \text{ N/mm}^2$$

$$k_M = \text{MIN}(t_1;t_2) * \frac{10}{\sqrt{\frac{M_{y,d}}{f_{h,d} * 10 * d_N}}} = 4,09$$

$$R_{D1} = 100 * f_{h,d} * t_1 * d_N = 2197,92 \text{ N}$$

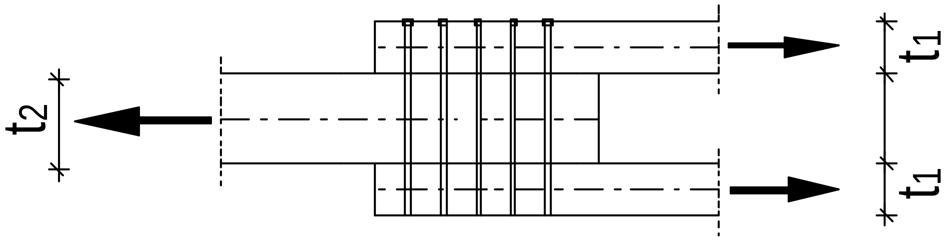
$$R_{D2} = 100 * 0,5 * f_{h,d} * t_2 * d_N = 1098,96 \text{ N}$$

$$R_{D3} = 100 * 0,367 * f_{h,d} * t_1 * d_N * \left(2 * \sqrt{1 + \frac{3}{k_M^{2}}} - 1\right) = 945,34 \text{ N}$$

$$R_{D4} = 155,6 * f_{h,d} * t_1 * \frac{d_N}{k_M} = 836,18 \text{ N}$$

$$R_D = 0.001 * \text{MIN}(R_{D1};R_{D2};R_{D3};R_{D4}) = \underline{\mathbf{0,84\ kN}}$$

Determination of required number of nails



System:

Timber thickness t_1 = 4,00 cm
Timber thickness t_2 = 4,00 cm

Materials:

Construction material CM =	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD =	SEL("wood/kmod"; CLD;)	=	permanent
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,60
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
γ_M =	1,30		
γ_S =	1,10		

Nails: 42x110

Nail length l_N = 12,00 cm
Nail diameter d_N = 0,42 cm

Load:

F_{sd} = 16,00 kN

Calculation:

Minimum timber thickness adhered to according to 6.3.1.2(11):
(MAX(7*d_N;(13*d_N -30)*ρ_k/400))/(MIN(t_1;t_2)) = 0,73 < 1
Minimum driving depth according to 6.3.1.2(4):
(8*d_N)/MIN(t_1;t_2) = 0,84 < 1
Minimum nail distances according to Tab.6.3.1.2:

a_1 =	10* d_N	=	4,20 cm
a_2 =	5 * d_N	=	2,10 cm
$a_{3,t}$ =	15* d_N	=	6,30 cm
$a_{4,t}$ =	5 * d_N	=	2,10 cm

pre-drilled $f_{h,k}$ = $0.082 \cdot \rho_k \cdot (1-0.1 \cdot d_N)$ = 29,85 N/mm²
not pre-drilled $f_{h,k}$ = $0.082 \cdot \rho_k \cdot (10 \cdot d_N)^{-0.3}$ = 20,26 N/mm²

$$f_{h,d} = \frac{k_{mod}}{g_M} \cdot f_{h,k} = 9{,}35\ \text{N/mm}^2$$

Characteristic value for yield moment of square nails

$$M_{y,d} = 270 \cdot \frac{(10 \cdot d_N)^{2,6}}{g_S} = 10242{,}81\ \text{Nmm}$$

Characteristic value for yield moment of round nails

$$M_{y,d} = 180 * \frac{(10 * d_N)^{2,6}}{\gamma_S} = 6828,54 \text{ Nmm}$$

$$k_M = MIN(t_1;t_2) * \frac{10}{\sqrt{\frac{M_{y,d}}{f_{h,d} * 10 * d_N}}} = 3,03$$

$$R_{D1} = 100 * f_{h,d} * t_1 * d_N = 1570,80 \text{ N}$$

$$R_{D2} = 100 * 0,5 * f_{h,d} * t_2 * d_N = 785,40 \text{ N}$$

$$R_{D3} = 36,7 * f_{h,d} * t_1 * d_N * \left(2 * \sqrt{1 + \frac{3}{k_M^2}} - 1\right) = 751,57 \text{ N}$$

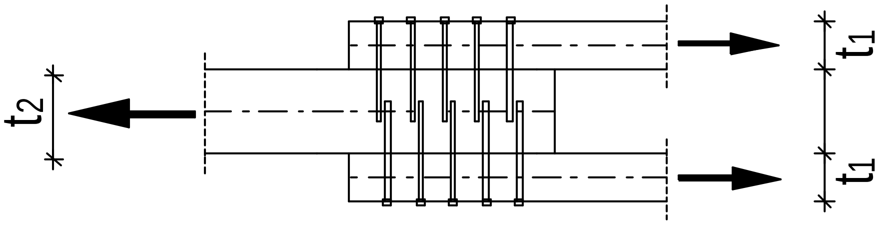
$$R_{D4} = 155,6 * f_{h,d} * t_1 * \frac{d_N}{k_M} = 806,66 \text{ N}$$

$$R_D = 0,001 * MIN(R_{D1};R_{D2};R_{D3};R_{D4}) = \underline{\mathbf{0,75\ kN}}$$

required number of nails n = $F_{sd}/(2*R_D)$ = **10,7**

selected: 2x6 nails 42x120 DIN 1151

Determination of required number of nails



System:

Timber thickness t_1 = 4,00 cm
Timber thickness t_2 = 4,00 cm
Timber width b = 8,00 cm

Materials:

Construction material CM =	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD =	SEL("wood/kmod"; CLD;)	=	permanent
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,60
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³

γ_M = 1,30
γ_S = 1,10

Nails: 42x110

Nail length l_N = 6,50 cm
Nail diameter d_N = 0,28 cm

Load:

F_{sd} = 16,00 kN

Calculation:

t_3 = $(l_N - t_1)$ = 2,50 cm

Checking the lapped joint length according to 6.3.1.2(10):
$4 * d_N / (t_2 - t_3)$ = 0,75 < 1

Minimum timber thickness adhered to according to 6.3.1.2(11):
$(MAX(7*d_N;(13*d_N-30)*\rho_k/400))/(MIN(t_1;t_2))$ = 0,49 < 1

Minimum driving depth according to 6.3.1.2(4):
$(8*d_N)/MIN(t_1;t_2;t_3)$ = 0,90 < 1

Minimum nail distances according to Tab.6.3.1.2:

a_1 = $10* d_N$ = 2,80 cm
a_2 = $5 * d_N$ = 1,40 cm
$a_{3,t}$ = $15* d_N$ = 4,20 cm
$a_{4,t}$ = $5 * d_N$ = 1,40 cm

pre-drilled $f_{h,k}$ = $0.082 * \rho_k * (1-0.1*d_N)$ = 30,29 N/mm²
not pre-drilled $f_{h,k}$ = $0.082 * \rho_k * (10*d_N)^{-0.3}$ = 22,88 N/mm²

$$f_{h,d} = \frac{k_{mod}}{g_M} * f_{h,k}$$ = 10,56 N/mm²

Characteristic value for yield moment of square nails

$M_{y,d}$ = $270 * \dfrac{(10 * d_N)^{2,6}}{\gamma_S}$ = 3569,29 Nmm

Characteristic value for yield moment of round nails

$M_{y,d}$ = $180 * \dfrac{(10 * d_N)^{2,6}}{\gamma_S}$ = 2379,53 Nmm

k_t = MAX(t_1;t_2;t_3)/MIN(t_1;t_2;t_3) = 1,60

k_M = $\text{MIN}(t_1;t_2;t_3) * \dfrac{10}{\sqrt{\dfrac{M_{y,d}}{f_{h,d} * 10 * d_N}}}$ = 2,79

R_{D1} = $100 * f_{h,d} * \text{MIN}(t_1;t_2;t_3) * d_N$ = 739,20 N

R_{D2} = $100*0,5*f_{h,d}*\text{MIN}(t_1;t_2;t_3)*d_N*(\sqrt{(3 * k_t^2 + 2 * k_t + 3)} - k_t - 1)$ = 416,02 N

R_{D3} = $36,7 * f_{h,d} * \text{MIN}(t_1;t_2;t_3) * d_N * \left(2 * \sqrt{1 + \dfrac{3}{k_M^2}} - 1\right)$ = 367,34 N

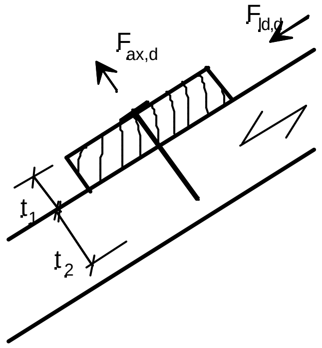
R_{D4} = $155,6 * f_{h,d} * \text{MIN}(t_1;t_2;t_3) * \dfrac{d_N}{k_M}$ = 412,26 N

R_D = $0,001 * \text{MIN}(R_{D1};R_{D2};R_{D3};R_{D4})$ = **0,37 kN**

required number of nails n = F_{sd}/R_D = **43,2**

selected: 2x3x8=48 nails 28x65 DIN 1151

Attachment of roof boarding/sheathing



System:

Timber thickness t_1 = 2,80 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
γ_M =	1,30		
γ_S =	1,10		

Nails: 38x100

Nail length l_N = 7,50 cm

Nail diameter d_N = 0,40 cm

Load:

Wind suction $F_{ax,d}$ = 0,55 kN CLD: short term

Roof shear Load scheme g $F_{la,d1}$ = 0,25 kN CLD: permanent

Roof shear Load scheme g+s $F_{la,d2}$ = 0,45 kN CLD: short term

Calculation:

Load-bearing capacity under shear load:

pre-drilled $f_{h,k}$ = $0.082 \cdot \rho_k \cdot (1-0.1 \cdot d_N)$ = 29,91 N/mm²

not pre-drilled $f_{h,k}$ = $0.082 \cdot \rho_k \cdot (10 \cdot d_N)^{-0.3}$ = 20,56 N/mm²

Characteristic value for yield moment of square nails

$M_{y,d}$ = $270 \cdot \frac{(10 \cdot d_N)^{2,6}}{g_S}$ = 9022,50 Nmm

Characteristic value for yield moment of round nails

$M_{y,d}$ = $180 \cdot \frac{(10 \cdot d_N)^{2,6}}{g_S}$ = 6015,00 Nmm

decisive load scheme:

Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	permanent
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow k_{mod1}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,60
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow k_{mod2}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90

$$\frac{F_{la,d1} * k_{mod2}}{F_{la,d2} * k_{mod1}} = 0{,}83 < 1$$

$\Rightarrow$ Load scheme g+s decisive

$$f_{h,d} = \frac{k_{mod2}}{g_M} * f_{h,k} = 14{,}23 \text{ N/mm}^2$$

$$t_2 = (l_N - t_1) = 4{,}70 \text{ mm}$$

$$k_t = MAX(t_1;t_2)/MIN(t_1;t_2) = 1{,}68$$

$$k_M = MIN(t_1;t_2) * \frac{10}{\sqrt{\frac{M_{y,d}}{f_{h,d} * 10 * d_N}}} = 2{,}72$$

$$R_{D1} = 100 * f_{h,d} * t_1 * d_N = 1593{,}76 \text{ N}$$

$$R_{D2} = 100 * 0{,}5 * f_{h,d} * t_2 * d_N = 1337{,}62 \text{ N}$$

$$R_{D3} = 100 * 0{,}367 * f_{h,d} * t_1 * d_N * \left(2 * \sqrt{1 + \frac{3}{k_M^{\;2}}} - 1\right) = 801{,}95 \text{ N}$$

$$R_{D4} = 155{,}6 * f_{h,d} * t_1 * \frac{d_N}{k_M} = 911{,}72 \text{ N}$$

$$R_D = 0{,}001 * MIN(R_{D1};R_{D2};R_{D3};R_{D4}) = \underline{0{,}80 \text{ kN}}$$

$$\frac{F_{la,d2}}{R_D} = \underline{0{,}56 < 1}$$

Pullout capacity:

according to DIN 1052-2 Section 6.3.1

$$f_{1,d} = 40 * 10^{-6} * \rho_K^2 * \frac{k_{mod2}}{g_M} = 4{,}00 \text{ N/mm}^2$$

$$f_{2,d} = 600 * 10^{-6} * \rho_K^2 * \frac{k_{mod2}}{g_M} = 59{,}98 \text{ N/mm}^2$$

$$R_{d1} = 100 * f_{1,d} * d_N * t_2 = 752{,}00 \text{ N}$$

$$R_{d2} = 100 * f_{2,d} * d_N^2 = 959{,}68 \text{ N}$$

$$R_d = MIN(R_{d1};R_{d2})/1000 = 0{,}75 \text{ kN}$$

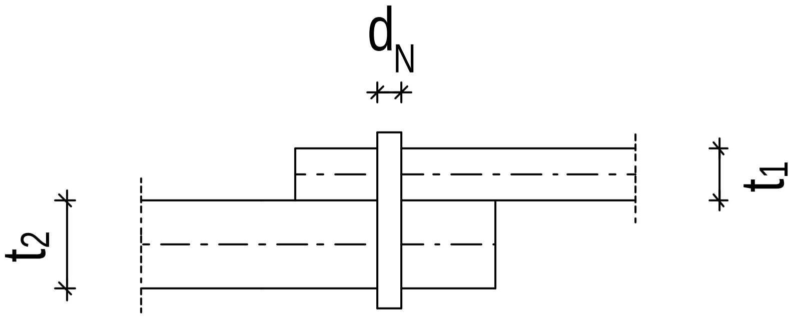
$$\frac{F_{ax,d}}{R_d} = \underline{0{,}73 < 1}$$

Combined loads:

$$\left(\frac{F_{ax,d}}{R_d}\right)^2 + \left(\frac{F_{la,d2}}{R_D}\right)^2 = \underline{0{,}85 < 1}$$

Nail calculation for a shear joint (thick plate)

Wood-metal connection with plate t_1 > 0.5 d



System:

Sheet thickness t_1 = 0,50 cm

Timber thickness t_2 = 4,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	S13
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³

γ_M = 1,30

γ_S = 1,10

Nails: 38x100

Nail length l_N = 10,00 cm

Nail diameter d_N = 0,38 cm

<u>Calculation:</u>

pre-drilled:

$f_{h,k} = 0.082 * \rho_k * (1-0.1*d_N)$ = 29,98 N/mm²

not pre-drilled::

$f_{h,k} = 0.082 * \rho_k * (10*d_N)^{-0.3}$ = 20,88 N/mm²

Characteristic value for yield moment of square nails

$$M_{y,d} = 270 * \frac{(10 * d_N)^{2,6}}{g_S} = 7896,02 \text{ Nmm}$$

Characteristic value for yield moment of round nails

$$M_{y,d} = 180 * \frac{(10 * d_N)^{2,6}}{g_S} = 5264,02 \text{ Nmm}$$

$$f_{h,d} = \frac{k_{mod}}{g_M} * f_{h,k} = 14,46 \text{ N/mm²}$$

$$R_{D1} = 110 * f_{h,d} * t_2 * d_N * \left(\sqrt{2 + \frac{4 * M_{y,d}}{f_{h,d} * 10^3 * d_N * t_2^2}} - 1\right) = 1200 \text{ N}$$

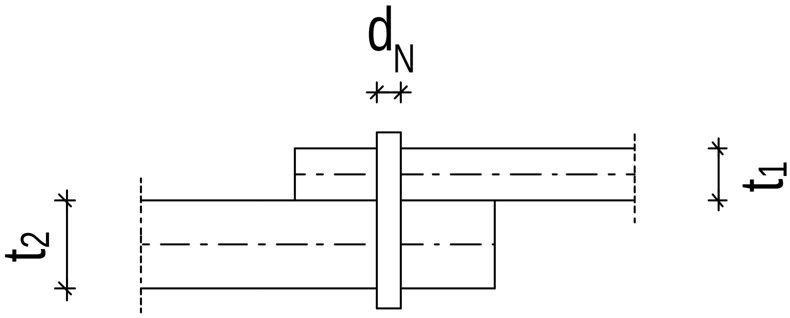
$$R_{D2} = 1,5 * \sqrt{20 * M_{y,d} * f_{h,d} * d_N} = 1141 \text{ N}$$

$$R_{D3} = 100 * f_{h,d} * t_2 * d_N = 2198 \text{ N}$$

R_D = 0.001 * MIN(R_{D1};R_{D2};R_{D3}) = **<u>1,14 kN</u>**

Nail calculation for a shear joint (thin plate) :

Wood-metal connection with plate t_1 < 0.5 d



System:

Sheet thickness t_1 =	0,15 cm
Timber thickness t_2 =	4,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	S13
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³

γ_M = 1,30

γ_S = 1,10

Nails: 38x100

Nail length l_N =		10,00 cm
Nail diameter d_N =		0,38 cm
t_1 / d_N	=	0,39 < 0,5

otherwise a different calculation applies

<u>Calculation:</u>

pre-drilled:

$f_{h,k}$ = 0.082 * ρ_k * (1-0.1*d_N) = 29,98 N/mm²

not pre-drilled::

$f_{h,k}$ = 0.082 * ρ_k * $(10*d_N)^{-0.3}$ = 20,88 N/mm²

Characteristic value for yield moment of square nails

$$M_{y,d} = 270 * \frac{(10 * d_N)^{2,6}}{g_S} = 7896,02 \text{ Nmm}$$

Characteristic value for yield moment of round nails

$$M_{y,d} = 180 * \frac{(10 * d_N)^{2,6}}{g_S} = 5264,02 \text{ Nmm}$$

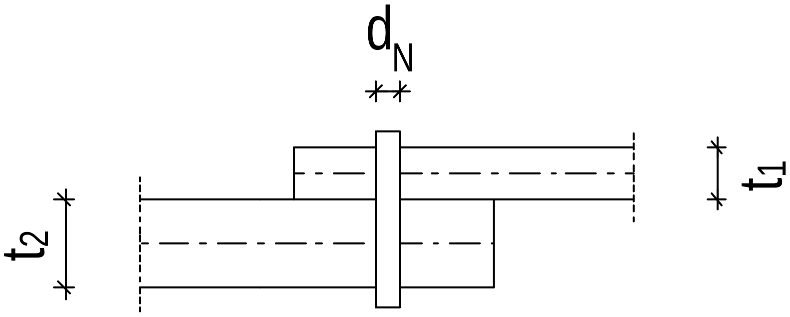
$f_{h,d}$ = k_{mod} / γ_M * $f_{h,k}$ = 14,46 N/mm²

R_{D1} = 100* (Ö2 -1)*$f_{h,d}$ * t_2 * d_N = 910,41 N

R_{D2} = 1,1 * Ö 20 * $M_{y,d}$ * $f_{h,d}$ * d_N = 836,65 N

R_D = 0.001 * MIN(R_{D1};R_{D2}) = **<u>0,84 kN</u>**

Nail calculation for a shear joint:



System:

Timber thickness t_1 = 4,00 cm

Timber thickness t_2 = 4,00 cm

Materials:

Construction material CM= SEL("wood/kmod"; CM;) = solid wood

Class of load duration CLD= SEL("wood/kmod"; CLD;) = short term

Utility class UC= SEL("wood/kmod"; UC;) = 1

$\Rightarrow k_{mod}$ = TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC) = 0,90

Strength grade SG= SEL("wood/EC"; SG;) = S10

ρ_k = TAB("wood/EC"; r k; SG=SG) = 380,00 kg/m³

γ_M = 1,30

γ_S = 1,10

Nails: 38x100

Nail length l_N = 10,00 cm

Nail diameter d_N = 0,38 cm

<u>Calculation:</u>

pre-drilled:

$f_{h,k}$ = 0.08 * ρ_k * (1-0.1*d_N) = 29,24 N/mm²

not pre-drilled::

$f_{h,k}$ = 0.082 * ρ_k * $(10*d_N)^{-0.3}$ = 20,88 N/mm²

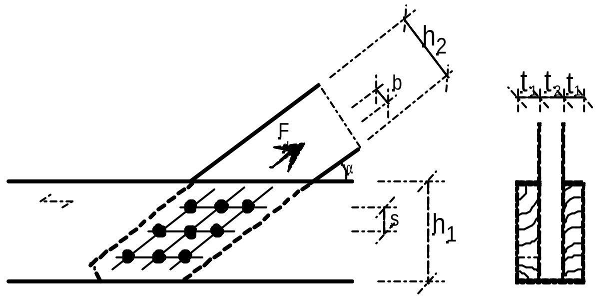
Characteristic value for yield moment of square nails

$M_{y,d}$ = 270 * $(10*d_N)^{2.6}$ / γ_S = 7896,02 Nmm

Characteristic value for yield moment of round nails

$M_{y,d}$ =	$180 * (d_N*10)^{2.6} / \gamma_S$	=	5264,02 Nmm
$f_{h,d}$ =	$k_{mod} / \gamma_M * f_{h,k}$	=	14,46 N/mm²
k_t =	$MAX(t_1;t_2)/MIN(t_1;t_2)$	=	1,00
k_M =	$MIN(t_1;t_2) * \dfrac{10}{\sqrt{\dfrac{M_{y,d}}{f_{h,d} * 10 * d_N}}}$	=	4,09
R_{D1} =	$100 * f_{h,d} * MIN(t_1;t_2) * d_N$	=	2197,92 N
R_{D2} =	$100 * 0,5 * f_{h,d} * MIN(t_1;t_2) * d_N * \left(\sqrt{3 * k_t^2 + 2 * k_t + 3} - k_t - 1\right)$	=	910,41 N
R_{D3} =	$100 * 0,367 * f_{h,d} * t_1 * d_N * \left(2 * \sqrt{1 + \dfrac{3}{k_M^2}} - 1\right)$	=	945,34 N
R_{D4} =	$155,6 * f_{h,d} * t_1 * \dfrac{d_N}{k_M}$	=	836,18 N
R_D =	$0.001 * MIN(R_{D1};R_{D2};R_{D3};R_{D4})$	=	**<u>0,84 kN</u>**

Tie rod connection



System:

Timber thickness t_1 =	3,80 cm
Timber thickness t_2 =	3,80 cm
Timber height h_1 =	14,00 cm
Timber height h_2 =	10,00 cm
Nail distance s =	3,50 cm
Nail distance b =	2,50 cm
Nail rows a =	3
Nail columns v =	3

Load:

F_d = 15 kN

Materials:

Construction material CM =	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD =	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
γ_M =	1,30		
γ_S =	1,10		

Nails: 42x110

Nail length l_N =	11,00 cm
Nail diameter d_N =	0,42 cm

Calculation:

$t_3 = (l_N - t_1 - t_2)$ = 3,40 cm

pre-drilled:

$f_{h,k} = 0.082 * \rho_k * (1\text{-}0.1{*}d_N)$ = 29,85 N/mm²

not pre-drilled::Imposed Load:

$f_{h,k} = 0.082 * \rho_k * (10{*}d_N)^{-0.3}$ = 20,26 N/mm²

Characteristic value for yield moment of square nails

$$M_{y,d} = 270 * \frac{(10 * d_N)^{2,6}}{g_S} = 10242,81 \text{ Nmm}$$

Characteristic value for yield moment of round nails

$$M_{y,d} = 180 * \frac{(10 * d_N)^{2,6}}{g_S} = 6828,54 \text{ Nmm}$$

$$f_{h,d} = \frac{k_{mod}}{\gamma_M} * f_{h,k} \qquad = \qquad 12,47\ \text{N/mm}^2$$

$$k_t = \text{MAX}(t_1;t_2)/\text{MIN}(t_1;t_2;t_3) \qquad = \qquad 1,12$$

$$k_M = \text{MIN}(t_1;t_2;t_3) * \frac{10}{\sqrt{\dfrac{M_{y,d}}{f_{h,d} * 10 * d_N}}} \qquad = \qquad 2,98$$

$$R_{D1} = 100 * f_{h,d} * \text{MIN}(t_1;t_2;t_3) * d_N \qquad = \qquad 1780,72\ \text{N}$$

$$R_{D2} = 100 * 0,5 * f_{h,d} * \text{MIN}(t_1;t_2;t_3) * d_N \qquad = \qquad 890,36\ \text{N}$$

$$R_{D3} = 36,7 * f_{h,d} * \text{MIN}(t_1;t_2;t_3) * d_N * \left(2 * \sqrt{1 + \frac{3}{k_M^2}} - 1\right) \qquad = \qquad 858,26\ \text{N}$$

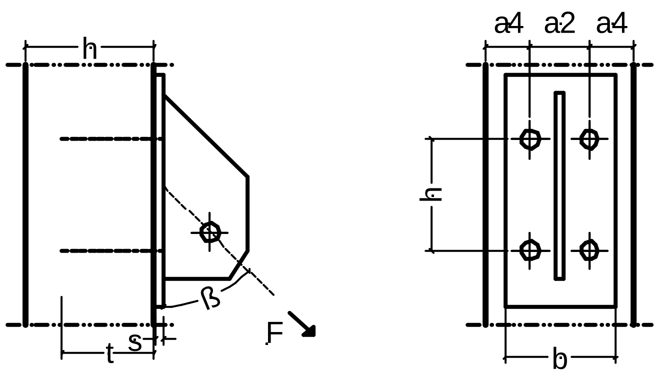
$$R_{D4} = 155,6 * f_{h,d} * \text{MIN}(t_1;t_2;t_3) * \frac{d_N}{k_M} \qquad = \qquad 929,80\ \text{N}$$

$$R_D = 0,001 * \text{MIN}(R_{D1};R_{D2};R_{D3};R_{D4}) \qquad = \qquad \underline{\mathbf{0,86\ kN}}$$

required number of nails n = $F_d/(2*R_D)$ = **8,7**

selected: 3x3=9 nails 42x110 DIN 1151

Wind brace connection:



System:

Beam thickness h =	16,00 cm
Driving depth t =	11,40 cm
Sheet thickness s =	0,60 cm
Angle of tensile force β =	45,00 °
Tensile force F_d =	23,54 kN

Materials: Post

Construction material CM =	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD =	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS14h
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	410,00 kg/m³
γ_M =	1,30		

Screws: 4 Sr Æ 12*120 DIN 571

Screw diameter d =	1,20 cm
$f_{u,k}$ =	300,00 N/mm²
γ_S =	1,10

Minimum distances Rod1 according to Tab.6.23:

a_1 =	7* d	=	8,40 cm
a_2 =	4 * d	=	4,80 cm
$a_{4,t}$ =	3 * d	=	3,60 cm

Design value of bearing stress resistance:

$$f_{h,d} = 0,082 * \rho_k * (1\text{-}0,1*d) * \frac{k_{mod}}{g_M} = 20,48 \text{ N/mm}^2$$

0.8/d = 0,67 < 1

⇒ calculation as for pins:

$$\frac{0,6*(t+s)+4*d}{t} = 1,05 > 1,0$$

⇒ d_{ef} = 9 *d = 10,80 mm

$$M_{y,d} = 0,8 * f_{u,k} * \frac{d_{ef}^{\,3}}{6 * g_S} = 45807,7 \text{ N/mm}$$

s/(d/2) = 1,00 < 1

⇒ thin plate

R_{D1} = $(\sqrt{2}-1)*f_{h,d}*(t-1,5*d)*d*100$ = 9772,52 N

R_{D2} = $1,1 * \sqrt{20 * M_{y,d} * f_{h,d} * d}$ = 5219,54 N

R_D = $0,001 * MIN(R_{D1};R_{D2})$ = **5,22 kN**

Design value for pullout:

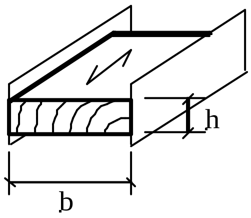
$f_{3,d}$ = $\frac{k_{mod}}{\gamma_M} * (1,5 + 6*d) * \sqrt{\rho_k}$ = 121,96 N/mm²

R_d = $f_{3,d} * (0,6*(t+s)-d)*0,01$ = **7,32 kN**

Combined loads:

$$\left(\frac{F_d * \sin(\beta)}{4 * R_d}\right)^2 + \left(\frac{F_d * \cos(\beta)}{4 * R_D}\right)^2 = \underline{\mathbf{0,96 < 1}}$$

Shrinkage and swelling of wood:



System:

The only appreciable stress is generated ⊥ to the fibre.

Width b = 32,00 cm

Height h = 12,00 cm

Coefficient for the degree of shrinkage and swelling β_{90} = 0,24 %

Strength grade SG= SEL("wood/EC"; SG;) = S10

$E_{90,mean}$ = TAB("wood/EC"; $E_{90.mean}$; SG=SG) = 370,00 N/mm²

Timber moisture before w_1 = 10,00 %

Timber moisture after w_2 = 15,00 %

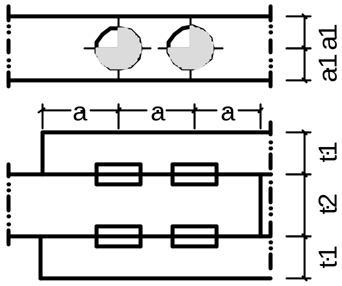
Stress:

$s_{c,90}$= $E_{90,mean}$*β_{90}*(w_2-w_1)/200 = 2,22 N/mm²

Total force:

$F_{s,90}$ = $s_{c,90}$ * b * h = 852,48 N

Tensile splice with special dowels:



System:

Distance a = 22,00 cm
Distance a_1 = 6,00 cm
Thickness t_1= 8,00 cm
Thickness t_2 = 10,00 cm
Cut surfaces n_z = 2
Tensile force $F_{t,d}$ = 95,00 kN

Construction material CM= SEL("wood/kmod"; CM;) = solid wood
Class of load duration CLD= SEL("wood/kmod"; CLD;) = normal
Utility class UC= SEL("wood/kmod"; UC;) = 2
⇒ k_{mod} = TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC) = 0,80

Transition to DIN 1052-2

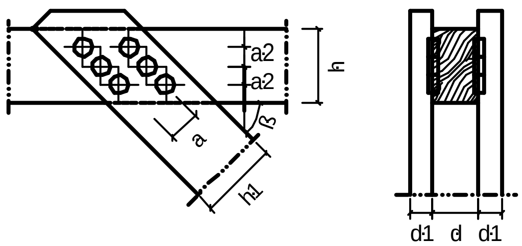
F_d = $F_{t,d}$/1.35 = 70,37 kN
Selection of dowel: Split ring connector Type A according to DIN 1052-2, Table 4, 6 or 7
Dowel ∅ 95 - A with 2 * M12 + 4 * washers 58/6
Number of dowels selected per cut n = 3
zul.N_c = 17,00 kN
b_{min}= 12,00 cm
a_{min}= 22,00 cm
a_{min4}= b_{min}/2 = 6,00 cm
a/a_{min} = **1,00 < 1**
a_1/a_{min4} = **1,00 < 1**

n_{ef} = MIN(2 + (1- $\frac{n}{20}$) * (n - 2);6) = 2,85

Structural verification:

$$\frac{F_d}{n_z * n_{ef} * zul.N_c} = \mathbf{0,73 < 1}$$

Diagonal tie connection with special dowels:



System:

Distance a = 8,00 cm
Distance a_2 = 9,00 cm
System angle β = 40,00 °
Beam height h = 30,00 cm
Beam thickness d = 16,00 cm
Strut height h_1 = 18,00 cm
Strut thickness d_1 = 8,00 cm
Number of rows n_z = 2
Tensile force $F_{t,d}$ = 165,00 kN

Construction material CM= SEL("wood/kmod"; CM;) = solid wood
Class of load duration CLD= SEL("wood/kmod"; CLD;) = normal
Utility class UC= SEL("wood/kmod"; UC;) = 2
⇒ k_{mod} = TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC) = 0,80

Transition to DIN 1052-2

F_d = $F_{t,d}$/1.35 = 122,22 kN
Selection of dowel: Split ring connector Type D according to DIN 1052-2, Table 4, 6 or 7 and 8
Dowel ∅ 65 - A with 6 * M12 + 12 * washers 58/6
Number of dowels selected per cut n = 6
per.N_c = 11,00 kN
Dowel height h_c = 2,70 cm
Dowel diameter d_c = 6,50 cm
Minimum wood dimensions of the beam from b/a = 110/40
Minimum wood dimensions of the strut from b/a = 100/40
Distance a_1 = 14,00 cm

$$a_{2,1} = d_c + \frac{h_c}{2} = 7,85 \text{ cm}$$

$a_{3,t}$ = 14,00 cm

$a_{4,strut}$ = 10/2 = 5,00 cm

$a_{4,beam}$ = 11/2 = 5,50 cm

$$l = \frac{a_2}{\sin(b)} = 14,00 \text{ cm}$$

a_1/l = **1,00 < 1**

$a_{2,1}/a_2$ = **0,87 < 1**

$$\frac{a_{4,beam}}{\frac{h}{2} - a_2} = \mathbf{0,92 < 1}$$

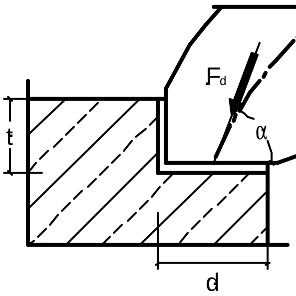
$$\frac{a_{4,strut}}{\frac{h_1 - a}{2}} = \underline{\mathbf{1{,}00 < 1}}$$

$$n_{ef} = \mathrm{MIN}\left(2 + \left(1 - \frac{n}{n_z} / 20 \right) * \left(\frac{n}{n_z} - 2 \right); 6 \right) = 2{,}85$$

Structural verification:

$$\frac{F_d}{2 * n_z * n_{ef} * per.N_c} = \underline{\mathbf{0{,}97 < 1}}$$

Arched beam:



System:

Cutting depth t =	12,00 cm
Support length d =	24,00 cm
Beam width b =	16,00 cm
Support angle a =	55,00 °

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS14k
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	410,00 kg/m³
Rectangular shear $f_{c,0,k}$ =	TAB("wood/EC"; fc,0.k; SG=SG)	=	27,50 N/mm²
Rectangular shear $f_{c,90,k}$ =	TAB("wood/EC"; fc,90.k; SG=SG)	=	5,50 N/mm²
g_M =	1,30		

Load:

F_d = 300,00 kN

Calculation:

$f_{c,0,d} = f_{c,0,k} * \frac{k_{mod}}{g_M}$ = 16,92 N/mm²

$f_{c,90,d} = f_{c,90,k} * \frac{k_{mod}}{g_M}$ = 3,38 N/mm²

Vertical supporting force:

$b = 90 - a$ = 35,00 °

$k_{c,b} = \frac{1}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin(b)^2 + \cos(b)^2}$ = 0,431

$V_{d,max} = 100 * b * d * k_{c,\beta} * f_{c,0,d} * 0,001$ = 280,03 kN

Horizontal supporting force:

$$k_{c,a} = \frac{1}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin(a)^2 + \cos(a)^2} = 0{,}271$$

$H_{d,max}$ = 100* b*d*$k_{c,\alpha}$*$f_{c,0,d}$*0.001 = 176,08 kN

$F_{d,h}$ = F_d * COS(a) = 172,07 kN

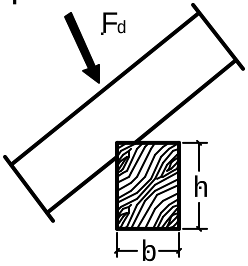
$F_{d,v}$ = F_d * SIN(a) = 245,75 kN

Structural verifications:

$$\frac{F_{d,h}}{H_{d,max}} = \underline{\mathbf{0{,}98 < 1}}$$

$$\frac{F_{d,v}}{V_{d,max}} = \underline{\mathbf{0{,}88 < 1}}$$

Centre purlin with 2-axis deflection:



Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
$f_{m,k}$ =	TAB("wood/EC"; fm.k; SG=SG)	=	24,00 N/mm²
g_M =	1,30		

Load:

Loading moments as a result of the sloping load

$M_{y,d}$ = 20,00 kNm

$M_{z,d}$ = 5,00 kNm

Calculation:

$f_{m,d} = f_{m,k} \cdot k_{mod} / \gamma_M$ = 16,62 N/mm²

Approximation $W_{y,req} = 1000 \cdot \dfrac{M_{y,d} + M_{z,d}}{f_{m,d}}$ = 1504,21 cm³

selected: Cross section:

b = 16,00 cm

h = 24,00 cm

$W_y = b \cdot \dfrac{h^2}{6}$ = 1536,00 cm³

$W_z = h \cdot \dfrac{b^2}{6}$ = 1024,00 cm³

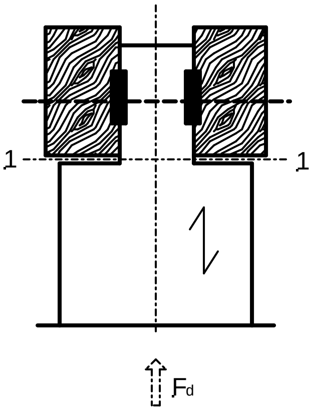
Structural verification:

k_m = 0,70 for rectangular cross section

otherwise k_m = 1.0

$$\left(\frac{M_{y,d}}{W_y} + k_m \cdot \frac{M_{z,d}}{W_z}\right) \cdot \frac{1000}{f_{m,d}} = \underline{\mathbf{0{,}99 < 1}}$$

Compression member connection



System:

Cross-sectional width in cut 1-1 b = 8,00 cm
Cross-sectional in cut 1-1 h = 16,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
Rectangular shear $f_{c,0,k}$ =	TAB("wood/EC"; fc,0.k; SG=SG)	=	21,00 N/mm²
g_M =	1,30		

Load:

F_d = 112,00 kN

Calculation:

A = b*h = 128,00 cm²

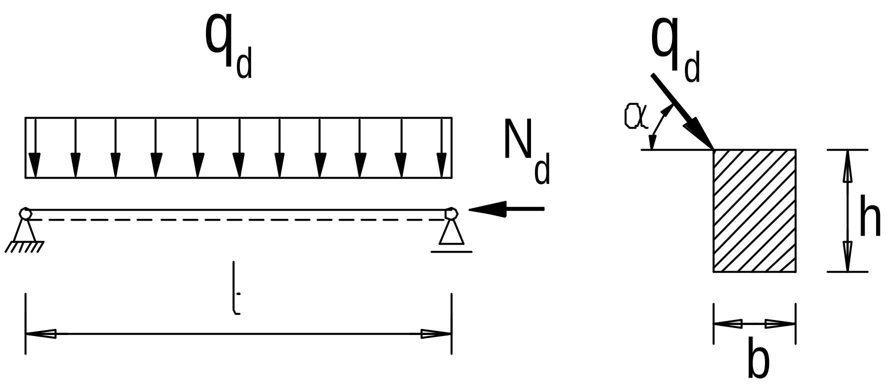
$f_{c,0,d} = f_{c,0,k} * \frac{k_{mod}}{g_M}$ = 12,92 N/mm²

$\sigma_{c,0,d} = \frac{F_d}{A} * 10$ = 8,75 N/mm²

Structural verification:

$\frac{s_{c,0,d}}{f_{c,0,d}}$ = **0,68 < 1**

Structural analysis of deflection in two directions and pressure:



System:

Beam width b = 18,00 cm
Beam height h = 24,00 cm
Beam length l = 3,50 m
Load angle α = 75,00 °

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
Rectangular shear $f_{c,0,k}$ =	TAB("wood/EC"; fc,0.k; SG=SG)	=	21,00 N/mm²
$f_{m,k}$ =	TAB("wood/EC"; fm.k; SG=SG)	=	24,00 N/mm²
g_M =	1,30		

Load:

q_d = 13,21 kN/m
N_d = 94,50 kN

Calculation:

$q_{d,y} = q_d * SIN(\alpha)$ = 12,76 kN/m

$q_{d,z} = q_d * COS(\alpha)$ = 3,42 kN/m

$M_{y,d} = q_{d,y} * \frac{l^2}{8}$ = 19,54 N/mm²

$M_{z,d} = q_{d,z} * \frac{l^2}{8}$ = 5,24 N/mm²

$f_{c,0,d} = f_{c,0,k} * \frac{k_{mod}}{g_M}$ = 12,92 N/mm²

$f_{m,d} = f_{m,k} * \frac{k_{mod}}{g_M}$ = 14,77 N/mm²

$W_y = b * \frac{h^2}{6}$ = 1728,00 cm³

$W_z = \quad h * \frac{b^2}{6} \quad = \quad 1296,00\ cm^3$

$A = \quad b * h / 1 \quad = \quad 432,00\ cm^2$

$k_m =$ 0,70 for rectangular cross section

otherwise $k_m = 1.0$

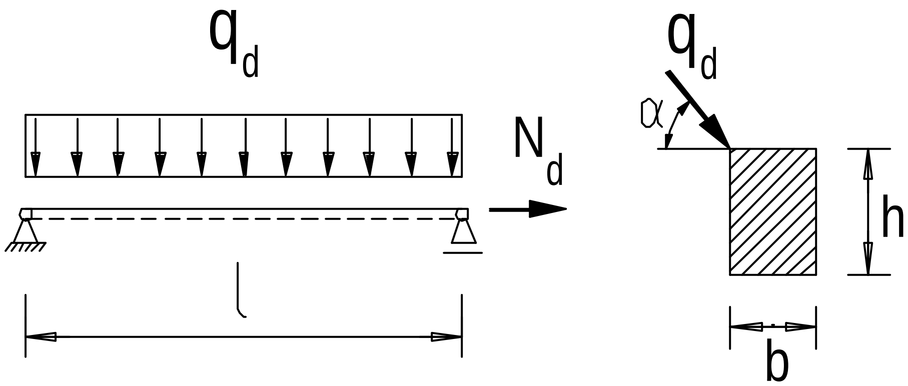
$\sigma_{c,0,d} = \quad 10 * N_d / A \quad = \quad 2,19\ N/mm^2$

Structural verifications:

$$\left(\frac{s_{c,0,d}}{f_{c,0,d}}\right)^2 + \left(\frac{M_{y,d}}{W_y} + k_m * \frac{M_{z,d}}{W_z}\right) * \frac{1000}{f_{m,d}} \quad = \quad \underline{\mathbf{0,99 < 1}}$$

$$\left(\frac{s_{c,0,d}}{f_{c,0,d}}\right)^2 + \left(k_m * \frac{M_{y,d}}{W_y} + \frac{M_{z,d}}{W_z}\right) * \frac{1000}{f_{m,d}} \quad = \quad \underline{\mathbf{0,84 < 1}}$$

Structural analysis of deflection in two directions and tension:



System:

Beam width b =	18,00 cm
Beam height h =	24,00 cm
Beam length l =	3,50 m
Load angle α =	75,00 m

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
$f_{t,0,k}$ =	TAB("wood/EC"; ft.0.k; SG=SG)	=	14,00 N/mm²
$f_{m,k}$ =	TAB("wood/EC"; fm.k; SG=SG)	=	24,00 N/mm²
g_M =	1,30		

Load:

q_d =	10,21 kN/m
N_d =	47,50 kN

Calculation:

$q_{d,y} = q_d * SIN(\alpha)$ = 9,86 kN/m

$q_{d,z} = q_d * COS(\alpha)$ = 2,64 kN/m

$M_{y,d} = q_{d,y} * \frac{l^2}{8}$ = 15,10 N/mm²

$M_{z,d} = q_{d,z} * \frac{l^2}{8}$ = 4,04 N/mm²

$f_{t,0,d} = f_{t,0,k} * \frac{k_{mod}}{g_M}$ = 8,62 N/mm²

$f_{m,d} = f_{m,k} * \frac{k_{mod}}{g_M}$ = 14,77 N/mm²

$W_y = b * \frac{h^2}{6}$ = 1728,00 cm³

$W_z = h * \frac{b^2}{6}$ = 1296,00 cm³

$A = b * h$ = 432,00 cm²

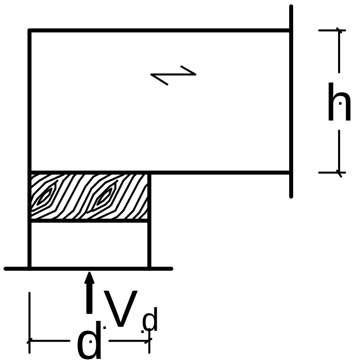
$k_m =$ 0,70 for rectangular cross section

otherwise $k_m = 1.0$

Structural verification:

$$\frac{10 * N_d}{A * f_{t,0,d}} + \left(\frac{M_{y,d}}{W_y} + k_m * \frac{M_{z,d}}{W_z}\right) * \frac{1000}{f_{m,d}}$$ = **0,87 < 1**

End support of a laminated timber beam



System:

Width of laminated timber beam b = 18,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS11
Rectangular shear $f_{c,90,k}$ =	TAB("wood/EC"; fc,90.k; SG=SG)	=	5,50 N/mm²
$f_{m,k}$ =	TAB("wood/EC"; fm.k; SG=SG)	=	24,00 N/mm²
g_M =	1,30		

Load:

V_d = 98,00 kN

Calculation:

$f_{c,90,d} = f_{c,90,k} * k_{mod} / g_M$ = 3,38 N/mm²

Assumption: l > 150 mm otherwise $k_{c,90}$ according to Tab. 5.1.5

$\Rightarrow k_{c,90}$ = 1,00

$$erf\_A = 10 * \frac{V_d}{f_{c,90,d} * k_{c,90}} = 289{,}94 \text{ cm}^2$$

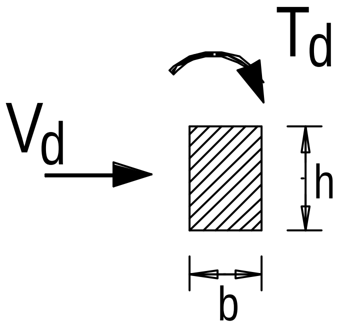
$$erf\_d = \frac{erf\_A}{b} = 16{,}11 \text{ cm}$$

sel. d = 17,00 cm

Structural verification:

erf_d / d = **0,95 < 1**

Stress analysis of shear force and torsion:



System:

Support width b = 30,00 cm
Support depth h = 14,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS16k
$f_{v,g,k}$ =	TAB("wood/EC"; fv.k; SG=SG)	=	2,70 N/mm²
g_M =	1,30		

Load:

relevant shear force V_d = 18,00 kN
relevant torsional moment T_d = 1,74 kNm

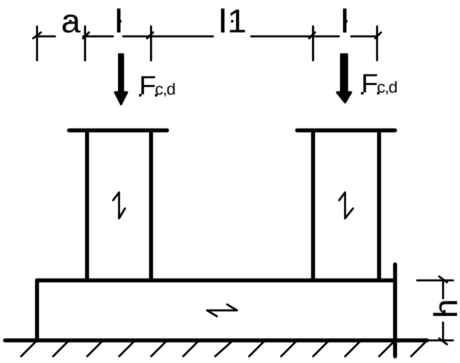
Calculation:

A =	b*h	=	420,00 cm²
η =	$1 + 0,6 * \frac{h}{b}$	=	1,28
$\tau_{tor,d}$ =	$3000 * T_d * \frac{h}{b * h^2}$	=	1,14 N/mm²
$\tau_{v,d}$ =	$30 * \frac{V_d}{2 * A}$	=	0,64 N/mm²
$f_{v,d}$ =	$k_{mod} * \frac{f_{v,g,k}}{g_M}$	=	1,87 N/mm²

Structural verifications:

Shear:	$\frac{t_{v,d}}{f_{v,d}}$	=	<u>**0,34 < 1**</u>
Torsion:	$\frac{t_{tor,d}}{f_{v,d}}$	=	<u>**0,61 < 1**</u>
Combination:	$\frac{t_{tor,d}}{f_{v,d}} + \left(\frac{t_{v,d}}{f_{v,d}}\right)^2$	=	<u>**0,73 < 1**</u>

Stud:



System:

Length of wood beam projection a =	6,00 cm
Support length l =	6,00 cm
Support depth d =	12,00 cm
clear distance between studs l_1 =	34,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	S10
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
$f_{c,90,k}$ =	TAB("wood/EC"; fc.90.k; SG=SG)	=	5,00 N/mm²
$f_{m,k}$ =	TAB("wood/EC"; fm.k; SG=SG)	=	24,00 N/mm²
g_M =	1,30		

Load:

$F_{c,d}$ = 20,00 kN

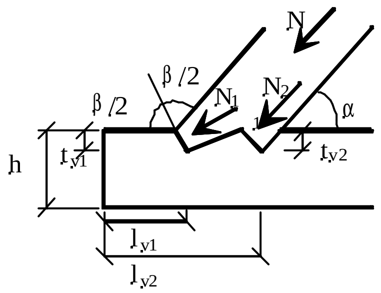
Calculation:

$f_{c,90,d}$ =	$k_{mod} * \frac{f_{c,90,k}}{g_M}$	=	3,46 N/mm²
für:	$15 / l_1$	=	0,44 < 1
und:	$l / 15$	=	0,40 < 1
$\Rightarrow$according to Tab. 5.1.5 $k_{c,90}$ =	$1 + a * \frac{150 - 10 * l}{1700}$	=	1,32
A =	$l * d$	=	72,00 cm²
$\sigma_{c,90,d}$ =	$10 * \frac{F_{c,d}}{A}$	=	2,78 N/mm²

Structural verification:

$$\frac{S_{c,90,d}}{k_{c,90} * f_{c,90,d}} = \underline{0,61 < 1}$$

Acceptable force of a double shoulder joint:



System:

Beam height h =	22,00 cm
Beam width b =	14,00 cm
Length of wood beam projection l_{v1} =	22,00 cm
Length of wood beam projection l_{v2} =	30,00 cm
Shoulder depth t_{v1} =	3,50 cm
Shoulder depth t_v =	4,50 cm
Angle α =	45,00 °

Comply with structural rules for shoulder depth and length of wood beam projection.

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S13
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	380,00 kg/m³
$f_{v,k}$ =	TAB("wood/EC"; fv.k; SG=SG)	=	2,50 N/mm²
$f_{c,0,k}$ =	TAB("wood/EC"; fc,0.k; SG=SG)	=	23,00 N/mm²
$f_{c,90,k}$ =	TAB("wood/EC"; fc.90.k; SG=SG)	=	5,00 N/mm²
g_M =	1,30		

Load:

N_d = 86,00 kN

Calculation:

$$f_{v,d} = f_{v,k} * \frac{k_{mod}}{g_M} = 1,54 \text{ N/mm}^2$$

$$f_{c,0,d} = f_{c,0,k} * \frac{k_{mod}}{g_M} = 14,15 \text{ N/mm}^2$$

$$f_{c,90,d} = f_{c,90,k} * \frac{k_{mod}}{g_M} = 3,08 \text{ N/mm}^2$$

with approximation:

$$k_F = \frac{1}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin(a)^2 * \cos(a) + \cos(a)^3} = 0,51$$

$$k_s = \frac{4}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin(a)^2 + \cos(a)^2 + 2 * \cos(a) + 1} = 0{,}77$$

$$l_{v1,min} = 10 * \frac{N_d}{2} * \frac{\cos(a)}{b * f_{v,d}} = 14{,}10 \text{ cm}$$

$$l_{v1,1} = 8 * t_v = 36{,}00 \text{ cm}$$

$$l_{v1,min} / l_{v1,1} = 0{,}39 < 1$$

$$l_{v1,min} / l_{v1} = \underline{0{,}64 < 1}$$

$$l_{v2,min} = 10 * \frac{N_d}{2} * \frac{\cos(a)}{b * f_{v,d}} = 14{,}10 \text{ cm}$$

$$l_{v2,1} = 8 * t_v = 36{,}00 \text{ cm}$$

$$\frac{l_{v2,min}}{l_{v2,1}} = 0{,}39 < 1$$

$$\frac{l_{v2,min}}{l_{v2}} = \underline{0{,}47 < 1}$$

$$R_{F,d} = b * t_v * f_{c,0,d} * k_F / 10 = \underline{45{,}46 \text{ kN}}$$

$$R_{S,d} = b * t_v * f_{c,0,d} * k_S / 10 = \underline{68{,}64 \text{ kN}}$$

detailed calculation:

$$k_{c,a} = \frac{1}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin(a)^2 + \cos(a)^2} = 0{,}358$$

$$R_{F,d} = k_{c,\alpha} * f_{c,0,d} * b * \frac{t_v}{\cos(a) * 10} = \underline{45{,}13 \text{ kN}}$$

$$k_{c,a} = \frac{1}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin\left(\frac{a}{2}\right)^2 + \cos\left(\frac{a}{2}\right)^2} = 0{,}655$$

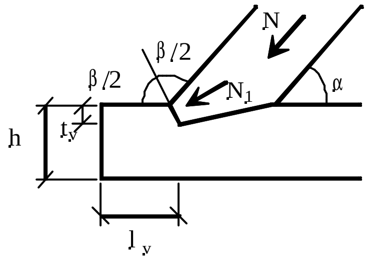
$$f_{c,0,5\alpha,d} = k_{c,\alpha} * f_{c,0,d} = 9{,}27 \text{ N/mm}^2$$

$$R_{S,d} = f_{c,0,5\alpha,d} * \alpha * \frac{b}{\cos\left(\frac{a}{2}\right)^2 * 100} = \underline{68{,}42 \text{ kN}}$$

Structural verification:

$$\frac{N_d}{R_{F,d} + R_{S,d}} = \underline{0{,}76 < 1}$$

Acceptable force of a face staggered joint:



System:

Beam height h =	22,00 cm
Beam width b =	14,00 cm
Length of wood beam projection l_v =	22,00 cm
Shoulder depth t_v =	4,50 cm
Angle α =	45,00 °

Comply with structural rules for shoulder depth and length of wood beam projection.

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S13
$f_{v,k}$ =	TAB("wood/EC"; fv.k; SG=SG)	=	2,50 N/mm²
$f_{c,0,k}$ =	TAB("wood/EC"; fc.0.k; SG=SG)	=	23,00 N/mm²
$f_{c,90,k}$ =	TAB("wood/EC"; fc.90.k; SG=SG)	=	5,00 N/mm²
g_M =	1,30		

Load:

N_d = 63,00 kN

Calculation:

$$f_{v,d} = f_{v,k} * \frac{k_{mod}}{g_M} = 1{,}54\ \text{N/mm}^2$$

$$f_{c,0,d} = f_{c,0,k} * \frac{k_{mod}}{g_M} = 14{,}15\ \text{N/mm}^2$$

$$f_{c,90,d} = f_{c,90,k} * \frac{k_{mod}}{g_M} = 3{,}08\ \text{N/mm}^2$$

with approximation:

$$k_s = \frac{4}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin(a)^2 + \cos(a)^2 + 2 * \cos(a) + 1} = 0{,}77$$

$$l_{v,min} = 10 * N_d * \frac{\cos(a)}{b * f_{v,d}} = 20{,}66\ \text{cm}$$

$$l_{v,1} = 8 * t_v = 36{,}00\ \text{cm}$$

$l_{v,min} / l_{v,1}$ = **0,57 < 1**

$l_{v,min} / l_v$ = **0,94 < 1**

$R_{S,d}$ = $b * t_v * f_{c,0,d} * k_S / 10$ = **68,64 kN**

detailed calculation:

$$k_{c,a} = \frac{1}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin\left(\frac{a}{2}\right)^2 + \cos\left(\frac{a}{2}\right)^2} = 0{,}655$$

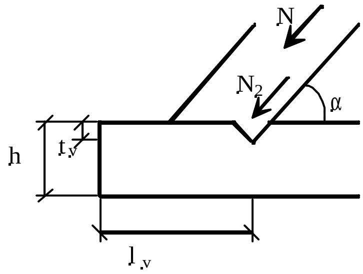
$f_{c,0,5\alpha,d}$ = $k_{c,\alpha} * f_{c,0,d}$ = 9,27 N/mm²

$$R_{S,d} = f_{c,0,5\alpha,d} * \alpha * \frac{b}{\cos\left(\frac{a}{2}\right)^2 * 100} = \mathbf{68{,}42\ kN}$$

Structural verification:

$$\frac{N_d}{R_{S,d}} = \mathbf{0{,}92 < 1}$$

Acceptable force of a heel staggered joint:



System:

Beam height h =	22,00 cm
Beam width b =	14,00 cm
Length of wood beam projection l_v =	22,00 cm
Shoulder depth t_v =	4,50 cm
Angle α =	45,00 °

Comply with structural rules for shoulder depth and length of wood beam projection.

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S13
$f_{v,k}$ =	TAB("wood/EC"; fv.k; SG=SG)	=	2,50 N/mm²
$f_{c,0,k}$ =	TAB("wood/EC"; fc.0.k; SG=SG)	=	23,00 N/mm²
$f_{c,90,k}$ =	TAB("wood/EC"; fc.90.k; SG=SG)	=	5,00 N/mm²
g_M =	1,30		

Load:

N_d = 43,00 kN

Calculation:

$$f_{v,d} = f_{v,k} * \frac{k_{mod}}{g_M} = 1,54 \text{ N/mm²}$$

$$f_{c,0,d} = f_{c,0,k} * \frac{k_{mod}}{g_M} = 14,15 \text{ N/mm²}$$

$$f_{c,90,d} = f_{c,90,k} * \frac{k_{mod}}{g_M} = 3,08 \text{ N/mm²}$$

with approximation:

$$k_F = \frac{1}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin(a)^2 * \cos(a) + \cos(a)^3} = 0,51$$

$$l_{v,min} = 10 * N_d * \frac{\cos(a)}{b * f_{v,d}} = 14,10 \text{ cm}$$

$$l_{v,1} = 8 * t_v = 36,00 \text{ cm}$$

$$l_{v,min} / l_{v,1} = 0,39 < 1$$

$$l_{v,min} / l_v = \underline{\mathbf{0,64 < 1}}$$

$R_{F,d} = \quad b * t_v * f_{c,0,d} * k_F / 10 \quad = \quad$ **45,46 kN**

detailed calculation:

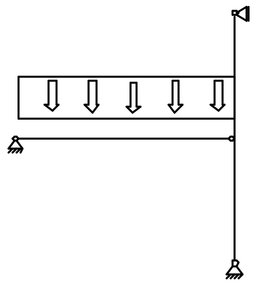
$$k_{c,a} = \frac{1}{\frac{f_{c,0,d}}{f_{c,90,d}} * \sin(a)^2 + \cos(a)^2} = 0{,}358$$

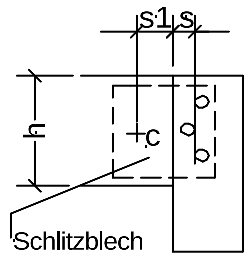
$$R_d = k_{c,\alpha} * f_{c,0,d} * b * \frac{t_v}{\cos(a) * 10} = \underline{\mathbf{45{,}13\ kN}}$$

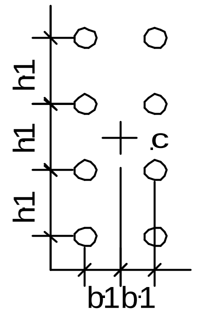
Structural verification:

$$\frac{N_d}{R_d} = \underline{\mathbf{0{,}95 < 1}}$$

Connection of a binding joist to a support:







System:

Centroidal distance s =	16,00 cm
Centroidal distance s_1 =	5,00 cm
Dowel distance b_1 =	5,00 cm
Dowel distance h_1 =	8,00 cm
Thickness of slotted plate m =	1,00 cm
Width of binding joist b =	16,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS11
ρ_k =	TAB("wood/EC"; ρk; SG=SG)	=	410,00 kg/m³
γ_M =	1,30		
γ_S =	1,10		

Load at tie point:

$V_{c,d}$ =			34,60 kN
$M_{c,d}$ =	$0{,}01 \cdot V_{c,d} \cdot (s_1+s)$	=	7,27 kNm

Geometric values:

$r_1 = \sqrt{b_1^2 + \left(\frac{h_1}{2}\right)^2} = 6{,}4$ cm

$r_2 = \sqrt{b_1^2 + (h_1 \cdot 1{,}5)^2} = 13{,}0$ cm

$\alpha_M = \operatorname{atan}\left(\frac{b_1}{h_1 \cdot 1{,}5}\right) = 22{,}62$ °

Calculation:

Drift pin forces:

as the result of shear force $F_{V,d} = \dfrac{V_{c,d}}{8}$ = 4,33 kN

as the result of moment $F_{M,d} = 100 * \dfrac{M_{c,d} * r_2}{4 * \left(r_1^{\,2} + r_2^{\,2}\right)}$ = 11,25 kN

Components:

$F_{M2,d,V} = F_{M,d} * \mathrm{SIN}(\alpha_M)$ = 4,33 kN

$F_{M2,d,H} = F_{M,d} * \mathrm{COS}(\alpha_M)$ = 10,38 kN

$F_{d,max} = \sqrt{\left(F_{M2,d,V} + F_{V,d}\right)^2 + F_{M2,d,H}^{\,2}}$ = 13,52 kN

$\alpha = \mathrm{atan}\left(\dfrac{F_{M2,d,V} + F_{V,d}}{F_{M2,d,H}}\right)$ = 39,84 °

selected: Dowel DPin Ø 16mm

$d =$ 1,60 cm

$f_{u,k} =$ 360,0 N/mm²

$M_{y,d} = 0{,}8 * f_{u,k} * \dfrac{d^3}{6 * \gamma_S}$ = 178,7 kNmm

$f_{h,0,d} = 0{,}082 * \rho_k * (1-0{,}1*d) * \dfrac{k_{mod}}{\gamma_M}$ = 17,38 N/mm²

$k_{90} = 1{,}35 + 0{,}15 * d$ = 1,59

$f_{h,1,d} = \dfrac{f_{h,0,d}}{k_{90} * \sin(\alpha)^2 + \cos(\alpha)^2}$ = 13,99 N/mm²

$t_1 = \dfrac{b - m}{2}$ = 7,50 cm

$R_{D1} = 100 * f_{h,1,d} * t_1 * d$ = 16788 N

$R_{D2} = 110 * f_{h,1,d} * t_1 * d * \left(\sqrt{2 + \dfrac{4 * M_{y,d}}{f_{h,1,d} * d * t_1^{\,2}}} - 1\right)$ = 11125 N

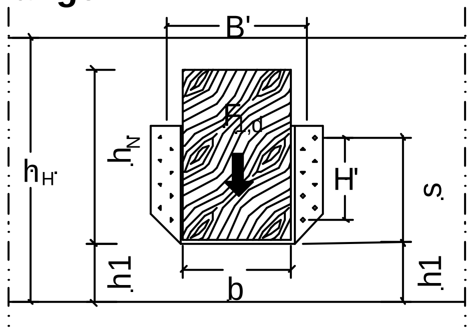
$R_{D3} = 150 * \sqrt{2 * M_{y,d} * f_{h,1,d} * d}$ = 13416 N

$R_D = 0{,}001 * \mathrm{MIN}(R_{D1};R_{D2};R_{D3})$ = <u>**11,13 kN**</u>

Structural verification:

$\dfrac{F_{d,max}}{2 * R_D}$ = <u>**0,61 < 1**</u>

Joist hanger:



System:

Height of principal beam h_H =	32,00 cm
Height of short-tie beam h_N =	24,00 cm
Joist hanger distance h_1 =	6,00 cm
Nail distance s =	15,20 cm

Load:

$F_{1,d}$ =	12,00 kN

Materials:

Principal beam: 120mm*380mm

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow$ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS11
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	410,00 kg/m³
$f_{t,90,g,k}$ =	TAB("wood/EC"; ft.90.k; SG=SG)	=	0,45 N/mm²
g_M =	1,30		

Short-tie beam: 140mm*240mm NH S 10

selected: GH joist hanger 04 140*160 with 14 RNä 4.0x60

Horizontal distance between centroidal axis of nails B' =	18,60 cm
Vertical Nail distance H' =	12,00 cm
Joist hanger thickness d_b =	0,20 cm
Nail diameter d_N =	0,40 cm
Nail length l_N =	6,00 cm
Number of nails n_N =	14
According to manufacturer's table $R_{0,d}$ =	16,40 kN

$$f = \frac{1}{1 - 0{,}93 \cdot \frac{s + h_1}{h_H}} = 2{,}61$$

$$t_{ef} = \mathrm{MIN}(12 \cdot d_N; l_N - d_b) = 4{,}80 \text{ cm}$$

$$f_{t,90,d} = f_{t,90,g,k} \cdot \frac{k_{mod}}{g_M} = 0{,}28 \text{ N/mm}^2$$

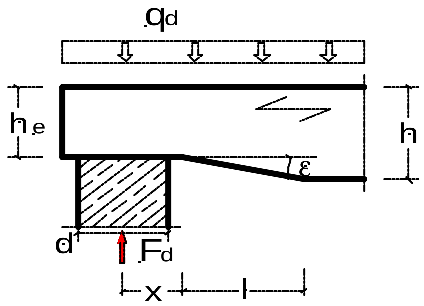
$$R_{t,90,d} = 0{,}0055 \cdot f \cdot (10 \cdot t_{ef})^{0,8} \cdot (10 \cdot h_H + 4 \cdot \sqrt{100 \cdot B' \cdot H'})^{0,8} \cdot f_{t,90,d} = 20{,}86 \text{ kN}$$

$$R_d = \mathrm{MIN}(R_{t,90,d}; R_{0,d}) = \underline{16{,}40 \text{ kN}}$$

Structural verification:

$$\frac{F_{1,d}}{R_d} = \underline{0{,}73 < 1}$$

Notch joist



System:

Cross-sectional width b =	14,00 cm
Support length d =	18,00 cm
Cross sectional height h =	88,00 cm
Cross sectional height at footing h_e =	78,00 cm
Notch distance x =	13,00 cm
Notch length l =	20,00 cm
Beam length l_1 =	850,00 cm

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	2
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS14k
ρ_k =	TAB("wood/EC"; r k; SG=SG)	=	410,00 kg/m³
$f_{v,g,k}$ =	TAB("wood/EC"; fv.k; SG=SG)	=	2,70 N/mm²
$f_{c,90,k}$ =	TAB("wood/EC"; fc.90.k; SG=SG)	=	5,50 N/mm²
for solid wood k_n =	IF(CM="Glulam";6,5;5)	=	6,50
g_M =			1,30

Load:

Supporting force F_d =	95,30 kN
Line load q_d =	2,80 kN/m

Calculation for splintering:

Maximum design shear force:

for uniformly distributed load k_r = $1 - 2 * \frac{h}{l_1}$ = 0,79

$F_{d,b}$ = $k_r * F_d$ = 75,29 kN

for individual load k_F =(0.5-h/l_1)*e/h with e the distance of the force from footing k

and $F_{b,d}$ =k_F * F_d

$f_{v,d}$ = $f_{v,g,k} * \frac{k_{mod}}{g_M}$ = 1,87 N/mm²

l=MAX(l;0.01)

ε = $\operatorname{atan}\left(\frac{h - h_e}{l}\right)$ = 26,57 °

$k_{\varepsilon} = \quad 1 + \frac{1{,}1 * \tan(\varepsilon)^{-1{,}5}}{\sqrt{h * 10}}$ = 1,105

$\alpha = \quad \frac{h_e}{h}$ = 0,886

$k_{90} = \quad \frac{k_n}{\sqrt{h * 10} * \left(\sqrt{\alpha * (1 - \alpha)} + 0{,}8 * \frac{x}{h} * \sqrt{\frac{1}{\alpha} - \alpha^2}\right)}$ = 0,566

$k_v = \quad \mathrm{MIN}(1;k_{90}*k_{\varepsilon})$ = 0,63

Structural verification:

$$\frac{15 * \frac{F_{d,b}}{b * h_e}}{k_v * f_{v,d}}$$ = **0,88 < 1**

Calculation of bearing pressure:

$f_{c,90,d} = \quad f_{c,90,k} * \frac{k_{mod}}{\gamma_M}$ = 3,81 N/mm²

Assumption: Support length t > 150 mm otherwise $k_{c,90}$ according to Tab. 5.1.5

$\Rightarrow k_{c,90} =$ 1,0

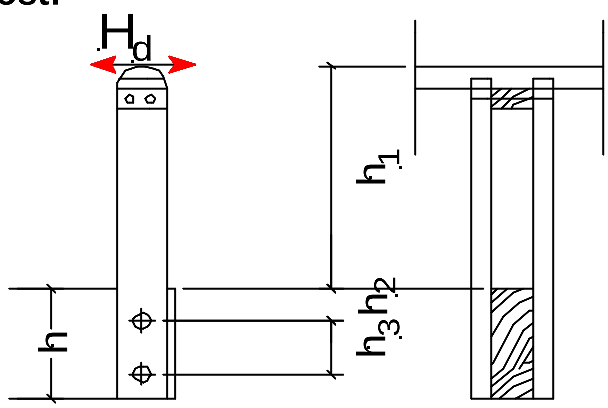
$erf\_A = \quad 10 * \frac{F_d}{f_{c,90,d} * k_{c,90}}$ = 250,13 N/mm²

$erf\_d = \quad \frac{erf\_A}{b}$ = 17,87 mm

Structural verification:

$$\frac{erf\_d}{d}$$ = **0,99 < 1**

Rail post:



System:

Post distance l =	1,70 m
Rail height h_1 =	100,00 cm
Dowel distance h_2 =	5,00 cm
Dowel distance h_3 =	23,00 cm
γ_Q =	1,50

Load: according to DIN 1055-3

H' =			1,00 kN/m
H_d =	γ_Q * l * H'	=	2,55 kN/m

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	Glulam
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	short term
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
⇒ k_{mod} =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,90
Strength grade SG=	SEL("wood/EC"; SG;)	=	BS11
$f_{t,90.g,k}$ =	TAB("wood/EC"; ft,90.k; SG=SG)	=	0,45 N/mm²
g_M =	1,30		

Calculation:

$M_{c,d}$ =	$0{,}01 * H_d * (h_1 + h_2 + \frac{h_3}{2})$	=	2,97 kNm
$F_{1,d}$ =	$\frac{H_d}{2} + \frac{M_{c,d}}{h_3 * 0{,}01}$	=	14,19 kN
$F_{2,d}$ =	$\frac{M_{c,d}}{h_3 * 0{,}01} - \frac{H_d}{2}$	=	11,64 kN

Transition to DIN 1052-2:

F_1 =	MAX($F_{1,d}$;$F_{2,d}$)/1,4	=	10,14 kN

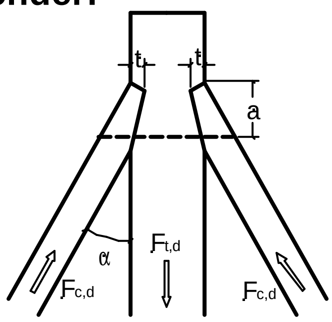
selected: 2 DPin Æ 85 - D

mit zul_N_c = 14,50 kN

Structural verification:

$$\frac{F_1}{zul\_N_c} = \underline{0{,}70 < 1}$$

Suspender:



System:

Height of diagonal brace h =	14,00 cm
Width of diagonal brace b =	14,00 cm
Height of suspender h_H =	18,00 cm
Width of suspender b_H =	14,00 cm
Shoulder depth t_v =	3,00 cm
Diameter of pin d =	1,20 cm
Angle α =	45,00 °

Comply with structural rules for shoulder depth and length of wood beam projection.

Materials:

Construction material CM=	SEL("wood/kmod"; CM;)	=	solid wood
Class of load duration CLD=	SEL("wood/kmod"; CLD;)	=	normal
Utility class UC=	SEL("wood/kmod"; UC;)	=	1
$\Rightarrow k_{mod}$ =	TAB("wood/kmod"; kmod; CM=CM; CLD=CLD; UC=UC)	=	0,80
Strength grade SG=	SEL("wood/EC"; SG;)	=	S13
$f_{t,0,k}$ =	TAB("wood/EC"; ft.0.k; SG=SG)	=	18,00 N/mm²
g_M =	1,30		

Calculation:

$A_{w,v}$ =	$2*b*t_v$	=	84,00 cm²
$A_{w,bo}$ =	$(d+1)*(h_H-2*t_v)$	=	26,40 cm²
A_w =	$b_H*h_H-MAX(A_{w,bo};A_{w,v})$	=	168,00 cm²
$f_{t,0,d}$ =	$f_{t,0,k} * \frac{k_{mod}}{g_M}$	=	11,08 N/mm²

Maximum force to be accepted by the construction:

$F_{t,d,max}$ =	$f_{t,0,d}*A_w*0,1$	=	186,14 kN