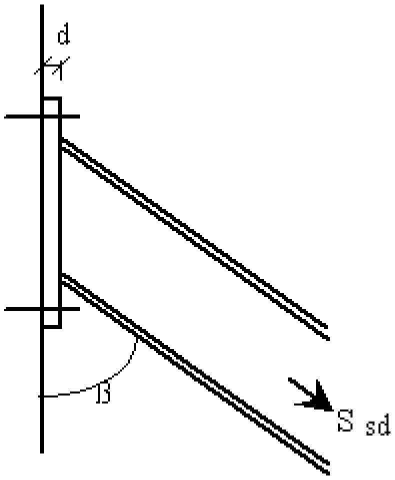
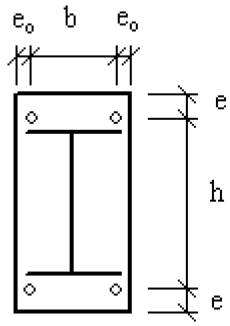
Euro-Code 3

Bolted connections

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 β =	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
γ_{M0}=	1,10		
γ_{Mb}=	1,25		

Loads:

S_{sd} = 424,26 kN

N_{sd} =	$COS(\beta) * S_{sd}$	=	300,00 kN
V_{sd} =	$SIN(\beta) * S_{sd}$	=	300,00 kN

Force per bolt:

$F_{t,sd}$ =	N_{sd} / 4	=	75,00 kN
$F_{v,sd}$ =	V_{sd} / 4	=	75,00 kN

Limit tension force of bolts:

$F_{t,Rd}$ =	$0,9 * f_{u,b} * A_s / \gamma_{Mb} / 10$	=	141,12 kN
$F_{t,sd} / F_{t,Rd}$		=	**<u>0,53 < 1</u>**

Check limit shear force for bolts:

$F_{v,Rd}$ =	$0,6 * f_{u,b} * \pi * d_l^2 / 4000 / \gamma_{Mb}$	=	120,64 kN
$F_{v,sd} / F_{v,Rd}$		=	**<u>0,62 < 1</u>**

Check combined:

$F_{t,sd} / (1,4 * F_{t,Rd}) + F_{v,sd} / F_{v,Rd}$ = **<u>1,00 ≤ 1</u>**

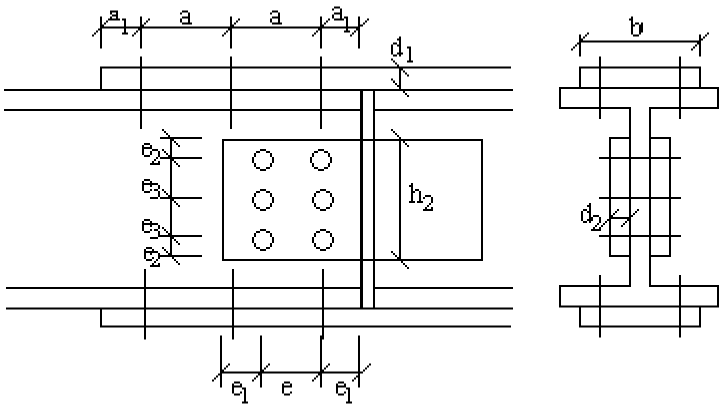
Analysis of bearing strength:

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

α =	$MIN(e / (3 * d_{ll}); b / (3 * d_{ll}) - 0,25; f_{u,b} / f_u; 1)$	=	0,53
Tab.6.5.3 $F_{b1,Rd}$ =	$2,5 * \alpha * f_u * d_l * d / \gamma_{Mb} / 10^3$	=	162,18 kN
6.5.5.(10) $F_{b2,Rd}$ =	$2/3 * F_{b1,Rd}$	=	108,12 kN
$F_{b,Rd}$ =	$F_{b2,Rd} + (F_{b1,Rd} - F_{b2,Rd}) * (e_o / d_l - 1,2) / 0,3$	=	162,18 kN

$F_{v,sd} / F_{b,Rd}$ = **<u>0,46 < 1</u>**

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

Flansch-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_{sd} =	157,95 kNm
V_{sd} =	60,75 kN

Check beam-web strength without holes:

$M_{c,Rd} = W_{pl} * f_y / \gamma_{M0} / 10_3$ = 78,91 kNm

$M_{sd} / M_{c,Rd}$ = <u>**2,00 < 1**</u>

$A_v = 1,04 * h * s / 100$ = 15,48 cm²

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

$V_{sd} / V_{pl,Rd}$ = <u>**0,32 < 1**</u>

Check beam-web strength with holes:

$A_z = 0,5 * A$ = 19,55 cm²

$A_{z,net} = A_z - (2 * d_{l1o} * t + ((n_s - 1) / 2) * d_{l1s} * s) / 100$ = 14,12 cm²

$(f_y / f_u * \gamma_{M2} / \gamma_{M0}) / (0,9 * A_{z,net} / A_z)$ = 1,14 ≤ 1

$W_{pl,net} = W_{pl} - 2 * d_{l1o} * t * (h - t) / 2000$ = 319,73 cm³

$M_{c,Rd,C} = W_{pl,net} * f_y / (\gamma_{M0} * 10^3)$ = 68,31 kNm

$M_{sd} / M_{c,Rd,C}$ = <u>**2,31 < 1**</u>

$A_{v,net} = A_v - n_s * d_{l1s} * s / 100$ = 12,13 cm²

$f_y / f_u * A_v / A_{v,net}$ = <u>**0,83 < 1**</u>

⇒ Area of holes will be neglected.

Analysis of bolts in flange:

$I_w = 2 / 12 * d_2 * h_2^3 / 10^4$ = 2926,93 cm⁴

$I_f = 2 * b_1 * d_1 * ((h + d_1) / 2)^2 / 10^4$ = 10816,00 cm⁴

$M_w = M_{sd} * I_w / (I_w + I_f)$ = 33,64 kNm

$M_f = M_{sd} * I_f / (I_w + I_f)$ = 124,31 kNm

Effective tension force in flange:

$N_{sd} = M_f / (h + d_1) * 10^3$ = 478,12 kN

Limit shear force

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

Limit bearing force:

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

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

$N_{sd} / MIN(F_{v,Rd}; F_{b,Rd})$ = <u>**1,32 < 1**</u>

Analysis of bolts in web:

n_{s1} =	$(n_s - 1) / 2 + 0,49$	=	1
n_{s2} =	$n_{s1} * 2$	=	2
m_{s1} =	$(m_s - 1) / 2 + 0,49$	=	1
m_{s2} =	$m_{s1} * 2$	=	2
I_p =	$(n_s * m_{s2} * ((m_s-1) / 2*e)^2 + n_{s2} * m_s * ((n_s-1) / 2*e_3)^2)/10^2$	=	420,00 cm²
M_{sw} =	$M_w + V_{sd} * e_3 / 10^3$	=	39,11 kNm
R_{sd} =	$\sqrt{}((10*M_{sw}*e_3/I_p)^2 + ((V_{sd}/(m_s*n_s)) + 10*M_{sw}*(m_s-1)/2*e/I_p)^2)$	=	96,27 kN
$F_{v,Rd}$ =	$m_s * 0,6 * f_{u,bs} * \pi/4 * d_{ls}^2 / 10^3 / \gamma_{M2}$	=	77,21 kN
α =	$MIN(e_1 / (3 * d_{l1s}); e / (3 * d_{l1s}) - 0,25; f_{u,bo} / f_u; 1)$	=	0,833
$F_{b,Rd}$ =	$2,5 * \alpha * f_u * d_{ls} * s / \gamma_{M2} / 10^3$	=	59,50 kN

$R_{sd} / MIN(F_{v,Rd}; F_{b,Rd})$ = **<u>1,62 < 1</u>**

Analysis of cover plates:

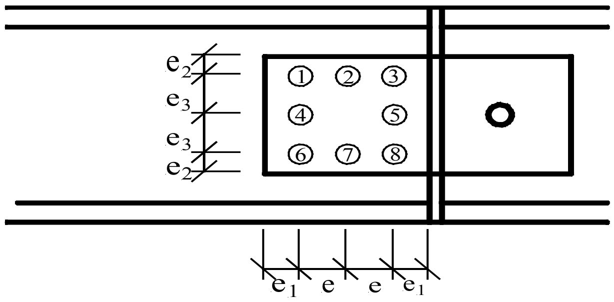
A =	$d_1 * b_1 / 100$	=	32,00 cm²
A_{net} =	$A - (2 * d_{l1o} * d_1) / 100$	=	23,20 cm²

Limit tension force:

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

$N_{sd} / N_{t,Rd}$ = **<u>0,80 < 1</u>**

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
IPE330 aus			
steel =	SEL("steel/EC"; Name;)	=	Fe 360
f_{u2} =	TAB("steel/EC"; fu; Name=steel)	=	360,00 N/mm²
γ_{Mb} =	1,25		
γ_{Mo} =	1,25		

Loads:

V_{sd} =	58,70 kN
M_{sd} =	910,00 kNcm

Analysis of the right bolt:

Check limit shear force

$F_{v,Rd} = 0,6 * \pi / 4 * (d_{l1} / 10)^2 * f_{u,b1} / \gamma_{Mo} / 10$ = 169,65 kN

$V_{sd} / F_{v,Rd}$ = **0,35 < 1**

Limit bearing strength

$F_{b,Rd} = 1,5 * d_{l1} * t * f_{u2} / \gamma_{Mb} / 10^3$ = 77,76 kN

$V_{sd} / F_{b,Rd}$ = **0,75 < 1**

Check bolts on left:

$I_p =$ $(6 * e^2 + 6 * e_3^2) / 100$ $=$ 444,00 cm²

Maximum horizontal force in bolt due to moment:

$F_h =$ $M_{sd} * e_3 / 10 / I_p$ $=$ 14,35 kN

Maximum vertical force:

$F_v =$ $M_{sd} * e / 10 / I_p + V_{sd} / n$ $=$ 17,59 kN

Resulting force

$R_r =$ $\sqrt{(F_h^2 + F_v^2)}$ $=$ 22,70 kN

$F_{v,Rd} =$ $0{,}6 * f_{u,b2} * d_{l2}^2 * \pi / 4 / \gamma_{Mb} / 10^3$ $=$ 31,86 kN

$R_r / F_{v,Rd}$ $=$ **0,71 < 1**

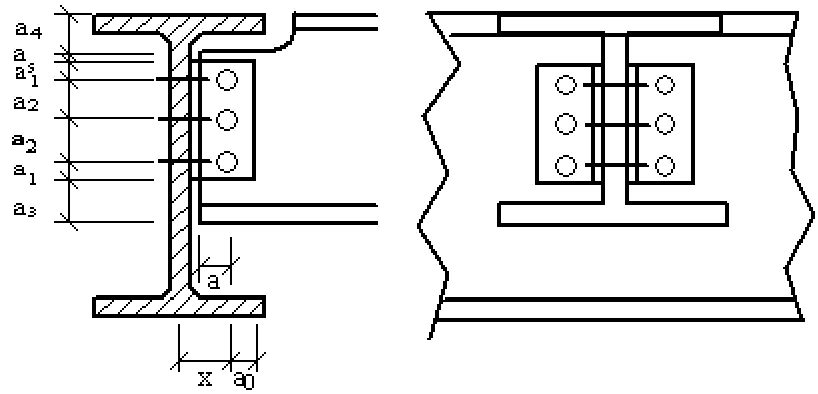
Analysis of bearing strength

$\alpha =$ $MIN(e_1 / (3 * d_{l2}); 2 * e / (3 * d_{l2}) - 0{,}25; f_{u,b2} / f_{u2}; 1)$ $=$ 0,90

$F_{b,Rd} =$ $2{,}5 * \alpha * f_{u2} * d_{l2} * t / \gamma_{Mb} / 10^3$ $=$ 50,54 kN

$F_h / F_{b,Rd}$ $=$ **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

γ_{Mb} =	1,25
γ_{M2} =	1,25
γ_{M0} =	1,10

Loads:

V_{sd} = 60,00 kN

Check bolt spacing as in 6.5.1:

$1{,}2 \cdot d_0 / a_1$	=	0,72 < 1
a_1 / MAX(12 * t; 150)	=	0,20 < 1
$1{,}5 \cdot d_0 / a_0$	=	0,77 < 1
a_0 / MAX(12 * t; 150)	=	0,23 < 1
$2{,}2 \cdot d_0 / a_2$	=	0,79 < 1
a_2 / MIN(12 * s; 200)	=	0,63 < 1

Check bolts strength:

Bolts in web of main beam $F_{v,sd}$ =	$V_{sd} / (2 \cdot n)$	=	10,00 kN
Bolts in web of connecting beam M_{sd} =	$x / 10 \cdot V_{sd}$	=	270,00 kNcm
Horizontal force due to M_{sd} $F_{h,sd}$ =	$M_{sd} / ((n - 1) \cdot a_2 / 10)$	=	27,00 kN
Distributed shearing force on bolts $F_{v,sd}$ =	V_{sd} / n	=	20,00 kN
Maximum bolt strength F_{sd} =	$\sqrt{(F_{h,sd}^2 + F_{v,sd}^2)}$	=	33,60 kN

Limit strength and analysis of bolts:

Bolts in web of main beam:

Plain ...in the shear plane $F_{v,Rd} = 0{,}6 \cdot f_{u,b} \cdot \pi \cdot d_l^2 / 4 / \gamma_{Mb} / 10^3$ = 38,60 kN

Limit bearing strength

$\alpha = \mathrm{MIN}(a_1 / (3 \cdot d_0);\ a_2 / (3 \cdot d_0) - 0{,}25;\ f_{u,b} / f_u;\ 1)$ = 0,556

$F_{b,Rd} = 2{,}5 \cdot \alpha \cdot f_u \cdot d_l \cdot t / \gamma_{Mb} / 10^3$ = 51,24 kN

Analysis:

$F_{sd} / F_{b,Rd}$ = **<u>0,66 < 1</u>**

Bolts in web of connecting beam:

$F_{v,Rd} = 2 \cdot 0{,}6 \cdot f_{u,b} \cdot \pi \cdot d_l^2 / 4 / \gamma_{Mb} / 10^3$ = 77,21 kN

Limit bearing strength

$\alpha = \mathrm{MIN}(a_1 / (3 \cdot d_0);\ a_2 / (3 \cdot d_0) - 0{,}25;\ f_{u,b} / f_u;\ 1)$ = 0,556

$F_{b,Rd} = 2{,}5 \cdot \alpha \cdot f_u \cdot d_l \cdot s / \gamma_{Mb} / 10^3$ = 42,27 kN

Analysis:

$F_{sd} / F_{b,Rd}$ = **<u>0,79 < 1</u>**

Check angle section strength

Design loads:

$V_{sdw} = V_{sd} / 2$ = 30,00 kN

$M_{sdw} = M_{sd} / 2$ = 135,00 kNm

$A = (2 \cdot a_2 + 2 \cdot a_1) \cdot t / 200$ = 6,40 cm²

$A_{net} = (2 \cdot a_2 + 2 \cdot a_1 - n \cdot d_0) \cdot t / 200$ = 4,24 cm²

$(f_y / f_u \cdot \gamma_{M2} / \gamma_{M0}) / (0{,}9 \cdot A_{net} / A)$ = 1,24 > 1

⇒ Subtract bottom bolt holes.

$A = A \cdot 2$ = 12,80 cm²

$A_{net} = A_{net} \cdot 2$ = 8,48 cm²

$f_y / f_u \cdot A / A_{net}$ = 0,99 < 1

⇒ Bolt holes should not be subtracted

$W_{pl} = (t \cdot (2 \cdot a_2 + 2 \cdot a_1)^2 / 4 - d_0 \cdot t \cdot a_2) / 1000$ = 44,00 cm³

$M_{pl,Rd} = W_{pl} \cdot f_y / 10 / \gamma_{M0}$ = 940,00 kNcm

$M_{sd} / M_{pl,Rd}$ = **<u>0,29 < 1</u>**

$V_{pl,Rd} = A \cdot f_y / 10 / (\sqrt{(3)} \cdot \gamma_{M0})$ = 157,88 kN

$V_{sd} / V_{pl,Rd}$ = **<u>0,38 < 1</u>**

Also:

$V_{sd} / (V_{pl,Rd} / 2)$ = **<u>0,76 < 1</u>**

⇒ No interaction.

Check shear failure:

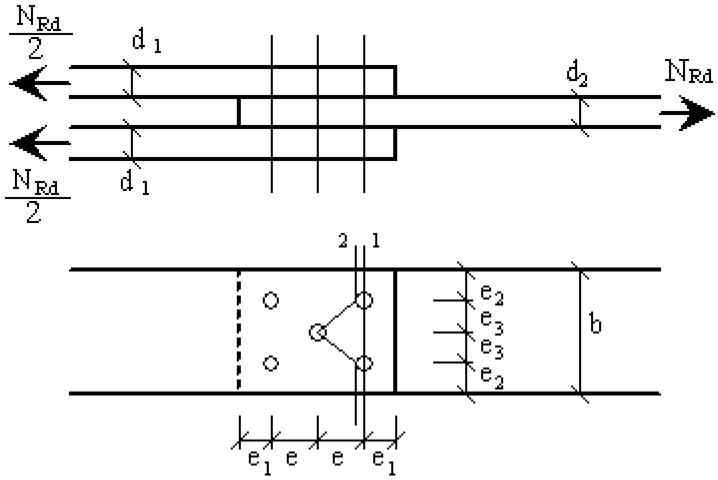
$L_v =$	$(n - 1) * a_2$	=	100,00 mm
$L_1 =$	a_2	=	50,00 mm
$L_1 / (5 * d_0)$		=	0,56 < 1
$L_2 =$	$(a - k * d_0) * f_u / f_y$	=	55,15 mm
$L_3 =$	$L_v + a_2 + a_3$	=	240,00 mm
$L_{v,eff} =$	$L_v + L_1 + L_2$	=	205,15 mm
$L_{v,eff} / L_3$		=	0,85 < 1
$L_3 / ((L_v + a_2 + a_3 - n * d_0) * f_u / f_y)$		=	0,84 < 1

Effective shear area:

$A_{v,eff} =$	$L_{v,eff} * t / 100$	=	16,41 cm²
$V_{eff,Rd} =$	$A_{v,eff} * f_y / 10 / \sqrt{(3)} / \gamma_{M0}$	=	202,41 kN

$V_{sd} / V_{eff,Rd}$ = **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
γ_{M0} = 1,10
γ_{M2} = 1,25

Loads:

N_{sd} = 100,00 kN

Joint with tension and shear:

Limit tension force of plate links

A = $b * d_1 * n_1 / 100$ = 25,20 cm²
d = $d_l + 2$ = 22,00 mm
Cross-section area 1:
$A_{net,1}$ = $A - 2 * d * d_1 * n_1 / 100$ = 19,92 cm²

Cross-section area 2:

$A_{net,2}$ =	A-3 *d *d_1 *n_1 /10² + 2 * e² / (4*e_3) * d_1 * n_1 / 10²	=	18,76 cm²
A_{net} =	MIN($A_{net,1}$; $A_{net,2}$)	=	18,76 cm²
$N_{t,Rd1}$ =	MIN(A * f_y / γ_{M0}; 0,9 * A_{net} * f_u / γ_{M2})/10	=	486,26 kN

Limit tension force of plate rechts

A =	b * d_2 * n_2 / 100	=	14,91 cm²
d =	d_l+ 2	=	22,00 mm
$A_{net,1}$ =	A - 2 * d * d_2 * n_2 / 100	=	11,79 cm²
$A_{net,2}$ =	A - 3 * d * d_2 * n_2 / 10² + 2 * e² / (4*e_3) * d_2 * n_2 / 10²	=	11,10 cm²
A_{net} =	MIN($A_{net,1}$; $A_{net,2}$)	=	11,10 cm²
$N_{t,Rd2}$ =	MIN(A * f_y / γ_{M0}; 0,9 * A_{net} * f_u / γ_{M2})/10	=	287,71 kN
$N_{t,Rd}$ =	MIN($N_{t,Rd2}$; $N_{t,Rd1}$)	=	**<u>287,71 kN</u>**

Limit tension force of boltsn:

A_s =	π * (d_l /10)² / 4	=	3,14 cm²
Thread in the shearing plane:			
$F_{v,Rd}$=	0,6 * n * (n_1 + n_2 - 1) * $f_{u,b}$ / 10* A_s / γ_{M2}	=	1507,20 kN
Limit bearing strength			
α =	MIN(e_1 / (3 * d); 2 * e / (3 * d) - 0,25; $f_{u,b}$ / f_u; 1)	=	0,91
$F_{b,Rd1}$ =	n_1 * n * 2,5 * α * f_u * d_l* d_1 / γ_{M2} / 10³	=	786,24 kN
α =	MIN(e_1 / (3 * d); 2 * e / (3 * d) - 0,25; $f_{u,b}$ / f_u; 1)	=	0,91
$F_{b,Rd2}$ =	n_2 * n * 2,5 * α * f_u * d_l* d_2 / γ_{M2} / 10³	=	465,19 kN
$F_{b,Rd}$ =	MIN($F_{b,Rd1}$; $F_{b,Rd2}$)	=	**<u>465,19 kN</u>**

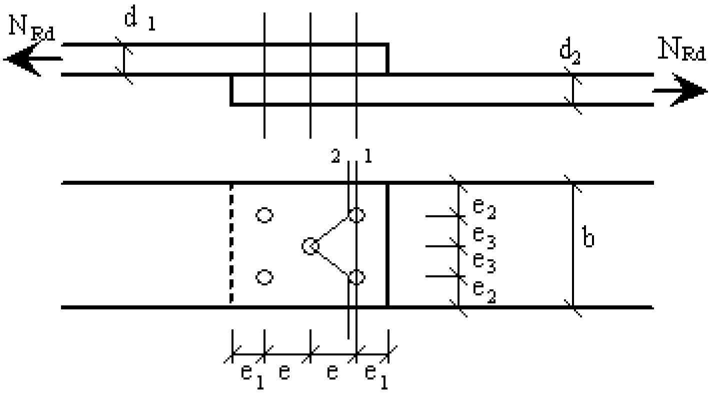
Maximum allowed force:

$N_{l,Rd}$ =	MIN($N_{t,Rd}$; $F_{v,Rd}$; $F_{b,Rd}$)	=	**<u>287,71 kN</u>**

Analysis:

N_{sd} / $N_{l,Rd}$ = **<u>0,35 < 1</u>**

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

Löcher mit normalem Lochspiel as in 6.5.8.1 (2) K_s =	1,00
Anzahl der Gleitfugen as in 6.5.8.1 (1) η =	1,00
Reibungszahl as in 6.5.8.3 μ =	0,40
as in 6.5.8.1 (3) γ_{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
γ_{M0} =	1,10		
γ_{M2} =	1,25		

Loads:

Tension force N_{sd} = 100,00 kN

Limit tension force of plate:

A =	b * MIN(d_1; d_2) / 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 * d * MIN(d_1; d_2) / 10²	=	15,36 cm²
Cross-section area 2:			
$A_{net,2}$ =	A - 3 * d * MIN(d_1; d_2)/10² + 2 * e² / (4*e_3) * MIN(d_1; d_2)/10²	=	15,85 cm²
A_{net} =	MIN($A_{net,1}$; $A_{net,2}$)	=	15,36 cm²

Friction-grip connection:

Plate

$N_{net,Rd}$ = $A_{net} * f_y / 10 / \gamma_{M0}$ = 328,15 kN

Limit tension force of bolts

Limit shank tension $F_{p,cd}$ = $0{,}7 * f_{u,b} / 10 * A_s$ = 212,10 kN

Limit slip resistance force $F_{s,Rd}$ = $n * K_s * \eta * \mu * F_{p,cd} / \gamma_{Ms}$ =339,36 kN

Limit bearing force:

α = $MIN(e_1 / (3 * d); 2 * e /(3 * d) - 0{,}25; f_{u,b} / f_u; 1)$ = 0,76

$F_{b,Rd}$ = $n * 2{,}5 * \alpha * f_u / 10 * d_l * MIN(d_1; d_2) / \gamma_{M2} / 100$ = 481,54 kN

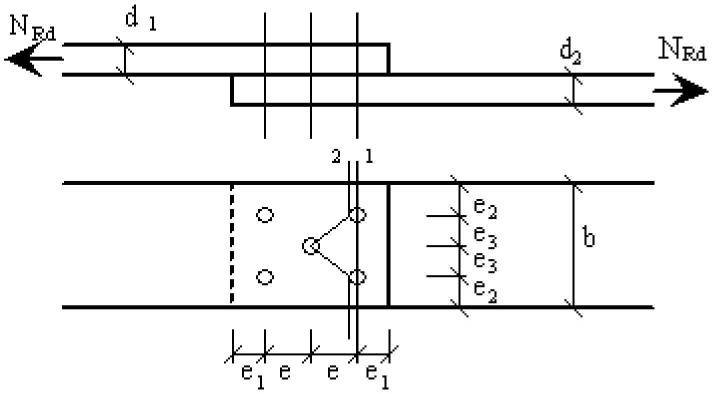
Maximum allowed force:

$N_{g,Rd}$ = $MIN(N_{net,Rd}; F_{b,Rd}; F_{s,Rd})$ = **328,15 kN**

Analysis:

$N_{sd} / N_{g,Rd}$ = **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
γ_{M0} =	1,10		
γ_{M2} =	1,25		

Loads:

N_{sd} = 100,00 kN

Joint with tension and shear:

Limit tension force of plate:

A =	b * MIN(d_1; d_2) / 100	=	19,20 cm²
d =	d_l+ 2	=	24,00 mm
Cross-section area 1:			
$A_{net,1}$ =	A - 2 * d * MIN(d_1; d_2) / 100	=	15,36 cm²
Cross-section area 2:			
$A_{net,2}$ =	A-3*d*MIN(d_1;d_2)/10² + 2*e²/(4*e_3)*MIN(d_1;d_2)/10²	=	15,85 cm²
A_{net} =	MIN($A_{net,1}$; $A_{net,2}$)	=	15,36 cm²
$N_{t,Rd}$ =	MIN(A * f_y / γ_{M0}; 0,9 * A_{net} * f_u / γ_{M2})/ 10	=	398,13 kN

Limit tension force of boltsn:

A_s =	π * (d / 10)² / 4	=	4,52 cm²
Thread in the shearing plane:			
$F_{v,Rd}$ =	0,6 * n * $f_{u,b}$ / 10 * A_s / γ_{M2}	=	433,92 kN/cm²
Limit bearing strength			
α =	MIN(e_1 / (3 * d); 2 * e / (3 * d) - 0,25;$f_{u,b}$ / f_u; 1)	=	0,76
$F_{b,Rd}$ =	n * 2,5 * α * f_u * d * MIN(d_1; d_2) / γ_{M2} / 10³	=	525,31 kN

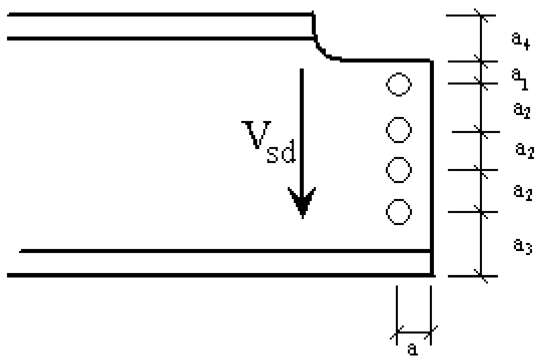
Maximum allowed force:

$N_{l,Rd}$ =	MIN($N_{t,Rd}$; $F_{v,Rd}$; $F_{b,Rd}$)	=	**398,13 kN**

Analysis:

N_{sd} / $N_{l,Rd}$		=	**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_{sd} = 58,70 kN

Limit shear force of section:

A_v =	1,04 * h * s / 100	=	29,95 cm²
$V_{pl,Rdp}$ =	A_v * f_y / 10 / √(3) / γ_{M0}	=	369,41 kN
$A_{v,net}$ =	A_v - (n * s * d_{l2}) / 100	=	22,91

$f_y / f_u * A_v / A_{v,net}$ = 0,85 < 1

⇒ Holes can be neglected.

Joint with shear failure as in Bild 6.5.5:

$L_v =$	$(n - 1) * a_2$	=	210,00 mm
$L_1 =$	a_1	=	43,00 mm
$L_1 / (5 * d_{l2})$		=	0,39 < 1
$L_2 =$	$(a - k * d_{l2}) * f_u / f_y$	=	59,74 mm
$L_3 =$	$L_v + a_1 + a_3$	=	328,00 mm
$L_{v,eff} =$	$L_v + L_1 + L_2$	=	312,74 mm
$L_{v,eff} / L_3$		=	0,95 < 1
$L_3 / (L_v + a_1 + a_3 - n + d_{l2})$		=	0,95 < 1

Effective shear area:

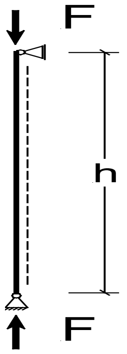
$A_{v,eff} =$	$L_{v,eff} * s / 100$	=	25,02 cm²
$V_{pl,Rd} =$	$A_{v,eff} * f_y / 10 / \sqrt{(3)} / \gamma_{M0}$	=	308,60 kN

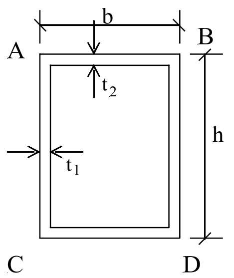
Maximum design force for shear:

$V_{Rd} =$	$MIN(V_{pl,Rdp}; V_{pl,Rd})$	=	**308,60 kN**

Columns

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 =	$\sqrt{(235 / f_y)}$	=	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
$b' / t_2 / (42 * \varepsilon)$		=	1,14 > 1

Section class 4

h' =	$h - 2 * t_2$	=	98,00 cm
$h' / t_1 / (42 * \varepsilon)$		=	2,33 > 1

Section class 4

Effective web area:

Part A-B; C-D As in Table 5.3.2:

ψ =		1,00	
k_σ =		4,00	

as in 5.3.5(3)

$\lambda_{trans,p}$ =	$b' / t_2 / (28{,}4 * \varepsilon * \sqrt{(k_\sigma)})$	=	0,85 >0,673
ρ =	$(\lambda_{trans,p} - 0{,}22) / \lambda_{trans,p}{}^2$	=	0,872
b_{eff} =	$\rho * b'$	=	41,86 cm

Part A-C; B-D As in Table 5.3.2:

ψ =			1,00 cm
k_σ =			4,00

as in 5.3.5(3)

$\lambda_{trans,p}$ =	$h' / t_1 / (28{,}4 * \varepsilon * \sqrt{(k_\sigma)})$	=	1,73 >0,673
ρ =	$(\lambda_{trans,p} - 0{,}22) / \lambda_{trans,p}{}^2$	=	0,505
h_{eff} =	$\rho * h'$	=	49,49 cm

Check buckling:

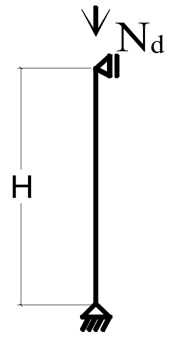
A_{eff} =	$2 * (h_{eff} * t_1 + b_{eff} * t_2) + 4 * t_1 * t_2$	=	186,70 cm²
A =	$b * h - (b - 2 * t_1) * (h - 2 * t_2)$	=	296,00 cm²
β_A =	A_{eff} / A	=	0,631
I_b =	$(b * h^3 - (b - 2 * t_1) * (h - 2 * t_2)^3) / 12$	=	401898,67 cm^4
I_h =	$(b^3 * h - (b - 2 * t_1)^3 * (h - 2 * t_2)) / 12$	=	138498,67 cm^4
I =	$MIN(I_b; I_h)$	=	138498,67 cm^4
N_{cr} =	$\pi^2 * E/10 * I / (H * 100)^2$	=	67941,82 kN
λ_{trans} =	$\sqrt{(\beta_A * A * f_y / 10 / N_{cr})}$	=	0,25

Apply strut curve b

α =			0,34
φ =	$0{,}5 * (1 + \alpha * (\lambda_{trans} - 0{,}2) + \lambda_{trans}{}^2)$	=	0,540
χ =	$1 / (\varphi + (\varphi^2 - \lambda_{trans}{}^2)^{0,5})$	=	0,9817
$N_{b,Rd}$ =	$\chi * \beta_A * A * f_y / 10 / \gamma_M$	=	3917,19 kN

$N_d / N_{b,Rd}$ = **0,893 < 1**

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 =	$\sqrt{(235 / f_y)}$	=	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:

$(h_1 / s) / (33 * \varepsilon)$ = 1,15 < 1

$(h_1 / s) / (38 * \varepsilon)$ = 1,00 < 1

Section class 2.

Flange:

$(b / 2 / t) / (10 * \varepsilon)$ = 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 :

λ_y =	l_y * 100 / i_y	=	30,00
λ_1 =	93,9 * ε	=	86,39
$\lambda_{trans,y}$ =	$\lambda_y / \lambda_1 * \sqrt{(\beta_A)}$	=	0,347

Apply strut curve a

As in Table 5.5.1 α =			0,21
φ =	$0,5 * (1 + \alpha * (\lambda_{trans,y} - 0,2) + \lambda_{trans,y}{}^2)$	=	0,576
χ =	$1 / (\varphi + (\varphi^2 - \lambda_{trans,y}{}^2)^{0,5})$	=	0,9655
$N_{b,y,Rd}$ =	$\chi * \beta_A * A * f_y /10/ \gamma_{M1}$	=	1298,60 kN

Check buckling In Z-Z Axis:

h / b		=	2,00 > 1,2
t / 10		=	1,07 cm < 4cm

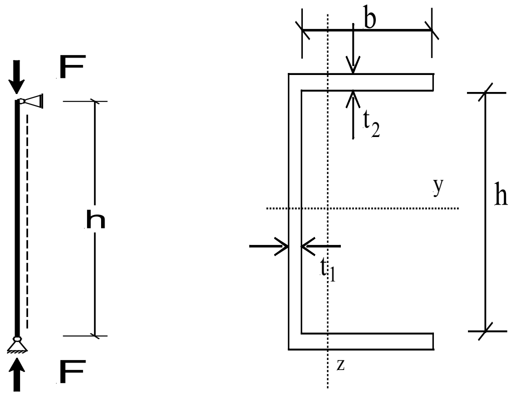
as in Tab 5.5.3 :

λ_z =	l_z * 100 / i_z	=	74,63
$\lambda_{trans,z}$ =	$\lambda_z / \lambda_1 * \sqrt{(\beta_A)}$	=	0,864

Apply strut curve b

As in Table 5.5.1 α =			0,34
φ =	$0,5 * (1 + \alpha * (\lambda_{trans,z} - 0,2) + \lambda_{trans,z}{}^2)$	=	0,986
χ =	$1 / (\varphi + (\varphi^2 - \lambda_{trans,z}{}^2)^{0,5})$	=	0,6844
$N_{b,z,Rd}$ =	$\chi * \beta_A * A * f_y /10/ \gamma_{M1}$	=	920,52 kN
max_N_d =	$MIN(N_{b,y,Rd}; N_{b,z,Rd})$	=	**<u>920,52 kN</u>**

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
ε =	$\sqrt{(235 / f_y)}$	=	1,00
λ_1 =	93,90 * ε	=	93,90

Partial safety factors:

γ_M = 1,10

Section classification As in Table 5.3.1:

$b / t_2 / (14 * \varepsilon)$ = 1,19 > 1

Section class 4

$h / t_1 / (33 * \varepsilon)$ = 0,76 > 1

Section class 1

Effective web area:

Flangee:

As in Table 5.3.3:

ψ = 1,00

k_σ = 0,43

as in 5.3.5(3)

$\lambda_{trans,p}$ = $b / t_2 / (28,4 * \varepsilon * \sqrt{k_\sigma})$ = 0,89 >0,673

ρ = $(\lambda_{trans,p} - 0,22) / \lambda_{trans,p}^2$ = 0,846

b_{eff} = $\rho * b$ = 16,92 cm

A_{eff} = $2 * (b_{eff} * t_2) + (h + 2 * t_2) * t_1$ = 79,49 cm²

Location of shear centre:

$y_{s,eff}$ = $(b_{eff} * (2 * t_2) * (b_{eff} + t_2) / 2) / A_{eff}$ = 4,628 cm

$b_{eff}^3 / 12 * (2 * t_2) + b_{eff} * (2 * t_2) * ((b_{eff} + t_1) / 2 - y_{s,eff})^2$ = 1766,44 cm⁴

$(h + 2 * t_2) * t_1^3 / 12 + (h + 2 * t_2) * t_1 * y_{s,eff}^2$ = 837,41 cm⁴

Iz,eff= 2603,85 cm⁴

$W_{z,eff}$ = $I_{z,eff} / ((b_{eff} + t_1) - y_{s,eff} - (t_1 / 2))$ = 201,97 cm³

Gross area:

A = $b * 2 * t_2 + (h + 2 * t_2) * t_1$ = 86,88 cm²

Location of center of gravity:

y_s = $(b * 2 * t_2 * (b + t_1) / 2) / A$ = 5,856 cm

I_z = $b^3/12*2*t_2+b*2*t_2*((b+t_1)/2-y_s)^2+(h+2*t_2)*t_1^3/12+(h+2*t_2)*t_1*y_s^2$ = 4018,23 cm⁴

e_{Nz} = $y_s - y_{s,eff}$ = 1,228 cm

Check buckling:

N_{cr} = $\pi^2 * E/10 * I_z / (H * 100)^2$ = 5205,15 kN

β_A = A_{eff} / A = 0,915

$\lambda_{trans,z}$ = $\sqrt{\beta_A * A * f_y /10 / N_{cr}}$ = 0,60

Apply strut curve c As in Table 5.5.3

α = 0,49

φ = $0,5 * (1 + \alpha * (\lambda_{trans,z} - 0,2) + \lambda_{trans,z}^2)$ = 0,778

χ = $1 / (\varphi + (\varphi^2 - \lambda_{trans,z}^2)^{0,5})$ = 0,7854

M_z = $N_d * e_{Nz} / 100$ = 8,60 kNm

ψ = 1,00

$\beta_{M\psi}$ = $1,8 - 0,7 * \psi$ = 1,10

μ_z = $\lambda_{trans,z} * (2 * \beta_{M\psi} - 4)$ = -1,080 < 0,9

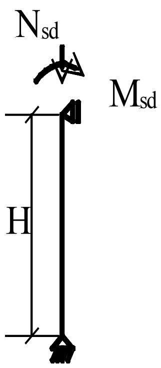
k_{z1} = $1 - (\mu_z * N_d) / (\chi * A_{eff} * f_y/10)$ = 1,515 ~ 1,5

k_{z2} = 1,500

k_z = $MIN(k_{z1}; k_{z2})$ = 1,500

$N_d / (\chi * A_{eff} * f_y/10 / \gamma_M) + 100 * k_z * M_z / (W_{z,eff} * f_y/10 / \gamma_M)$ = **<u>0,824 < 1</u>**

Column with compression and moment:



Load diagram:

Column heigth H = 5,00 m

Loads:

N_{sd} =	200,00 kN
$M_{y,sd}$ =	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
ε =	$\sqrt{(235 / f_y)}$	=	1,00

Partial safety factors:

γ_M =	1,10
γ_g =	1,35
platischer Formbeiwert α_{ply} =	1,14
platischer Formbeiwert α_{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} =	$\alpha_{ply} * W_{ely}$	=	250,80 cm³
W_{elz}=	TAB("steel/"Typ; Wz; Name=Profil)	=	76,90 cm³
W_{plz} =	$\alpha_{plz} * W_{elz}$	=	96,13 cm³

Section classification As in Table5.3.1:

Web will be assumed as pressed in

$(h_1 / s) / (33 * \varepsilon)$ = 0,53 < 1

Section class 1.

Flange:

$(b / 2 / t) / (10 * \varepsilon)$ = 0,89 < 1

Section class 1.

Check bending and buckling:

as in 5.5.1.1 (1) β_A = 1,00 Cross-section class 1

as in 5.1.1 (2) γ_{M1} = 1,10 Cross-section class 1

λ_y = H * 100 / i_y = 76,10

λ_1 = 93,9 * ε = 93,90

$\lambda_{trans,y}$ = λ_y / λ_1 * $\sqrt{(\beta_A)}$ = 0,810

λ_z = 100 * H / i_z = 125,63

$\lambda_{trans,z}$ = λ_z / (π * $\sqrt{(E / f_y)}$) = 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

α = 0,34

φ = 0,5 * (1 + α * ($\lambda_{trans,y}$ - 0,2) + $\lambda_{trans,y}{}^2$) = 0,932

χ_y = 1 / (φ + (φ^2 - $\lambda_{trans,y}{}^2)^{0,5}$) = 0,7179

Apply strut curve c for z-Achse

α = 0,49

φ = 0,5 * (1 + α * ($\lambda_{trans,z}$ - 0,2) + $\lambda_{trans,z}{}^2$) = 1,677

χ_z = 1 / (φ + (φ^2 - $\lambda_{trans,z}{}^2)^{0,5}$) = 0,3724

ψ = 0,00

β_{My} = 1,8 - 0,7 * ψ = 1,80

μ_y = $\lambda_{trans,y}$ * (2 * β_{My} - 4) + (W_{ply} - W_{ely}) / W_{ely} = -0,184 < 0,9

k_y = 1 - (μ_y * N_{sd}) / (χ_y * A * f_y) = 1,006 < 1,5

χ = MIN(χ_y; χ_z) = 0,372

N_{sd} / (χ * A * f_y / 10 / γ_M) + 100 * k_y * $M_{y,sd}$ / (W_{ply} * f_y / 10 / γ_M) = **0,836 < 1**

Check torsional-flexural buckling:

β_w = 1,00 Cross-section class 1

As in Table: F.1.1

ψ = 0,00

k = 1,00

C_1 = 1,879

λ_{LT} = 90 * H / i_z / ((C_1)^{0,5} * (1 + 1 / 20 * ((100 * H / i_z) / (h / t))²)^{0,25}) = 59,208

$\lambda_{trans,LT}$ = λ_{LT} / λ_1 * $\sqrt{(\beta_w)}$ = 0,631

For rolled sections strut curve a

α = 0,21

φ = 0,5 * (1 + α * ($\lambda_{trans,LT}$ - 0,2) + $\lambda_{trans,LT}{}^2$) = 0,74

χ_{LT} = 1 / (φ + (φ^2 - $\lambda_{trans,LT}{}^2)^{0,5}$) = 0,89

$\beta_{M,LT}$ = 1,8 - 0,7 * ψ = 1,80

$\lambda_{trans,z}$ = λ_z / λ_1 * $\sqrt{(\beta_A)}$ = 1,338

μ_{LT} = 0,15 * $\lambda_{trans,z}$ * $\beta_{M,LT}$ - 0,15 = 0,211 < 0,9

h / b = 0,95 < 1,2

t / 10 = 0,90 cm < 10cm

Z-Z axis Strut curve c

α = 0,49

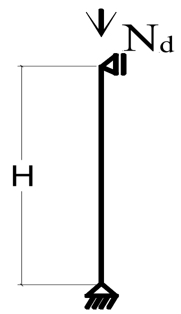
φ = 0,5 * (1 + α * ($\lambda_{trans,z}$ - 0,2) + $\lambda_{trans,z}{}^2$) = 1,674

χ_z = 1 / (φ + (φ^2 - $\lambda_{trans,z}{}^2)^{0,5}$) = 0,3731

k_{LT} = 1 - (μ_{LT} * N_{sd}) / (χ_z * A * f_y) = 0,988 < 1,0

N_{sd} / (χ_z * A * f_y / 10 / γ_M) + 100 * k_{LT} * $M_{y,sd}$ / (χ_{LT}*W_{ply} * f_y / 10 / γ_M) = **0,854 < 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 =	$\sqrt{(235 / f_y)}$	=	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 cm^4
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:

$(h_1 / s) / (33 * \varepsilon)$ = 0,57 < 1

Flange:

$(b / 2 / t) / (10 * \varepsilon)$ = 0,79 < 1

Section class 1.

Analysis:

β_A =	1,00 Cross-section class 1		
γ_{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
λ_y =	k_y * H * 100 / i_y	=	61,54
λ_1 =	93,9 * ε	=	93,90
$\lambda_{trans,y}$ =	$\lambda_y / \lambda_1 * \sqrt{(\beta_A)}$	=	0,655
Apply strut curve b			
As in Table 5.5.1 α =			0,34
φ =	$0,5 * (1 + \alpha * (\lambda_{trans,y} - 0,2) + \lambda_{trans,y}^2)$	=	0,792
χ =	$1 / (\varphi + (\varphi^2 - \lambda_{trans,y}^2)^{0,5})$	=	0,8083
$N_{b,Rd}$ =	$\chi * \beta_A * A * f_y / 10 / \gamma_{M1}$	=	2572,97 kN

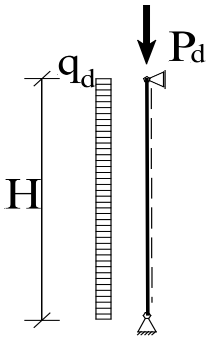
$N_d / N_{b,Rd}$ = **0,78 < 1**

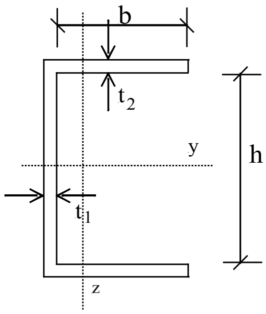
Check buckling In Z-Z Axis:

h / b		=	1,00 < 1,2
t / 10		=	1,90 cm < 10cm
λ_z =	k_z * H * 100 / i_z	=	73,88 m
$\lambda_{trans,z}$ =	$\lambda_z / \lambda_1 * \sqrt{(\beta_A)}$	=	0,787
Apply strut curve c			
As in Table 5.5.1 α =			0,49
φ =	$0,5 * (1 + \alpha * (\lambda_{trans,z} - 0,2) + \lambda_{trans,z}^2)$	=	0,953
χ =	$1 / (\varphi + (\varphi^2 - \lambda_{trans,z}^2)^{0,5})$	=	0,6709
$N_{b,Rd}$ =	$\chi * \beta_A * A * f_y / 10 / \gamma_{M1}$	=	2135,60 kN

$N_d/N_{b,Rd}$ = **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
ε =	$\sqrt{(235 / f_y)}$	=	1,00		
λ_1 =	93,90 * ε	=	93,90		

Partial safety factors:

γ_M = 1,10

Section classification:

As in Table 5.3.1

$b / t_2 / (14 * \varepsilon)$ = 1,13 > 1

Section class 4

As in Table 5.3.2

$h / t_1 / (33 * \varepsilon)$ = 0,89 > 1

Section class 1

A =	$b * 2 * t_2 + (h + 2 * t_2) * t_1$	=	42,00 cm²

Location of center of gravity:

y_s =	$(b * 2 * t_2 * (b + t_1) / 2) / A$	=	1,425 cm

$b^3 / 12 * 2 * t_2 + b * 2 * t_2 * ((b + t_1) / 2 - y_s)^2$	=	252,53 cm^4
$(h + 2 * t_2) * t_1^3 / 12 + (h + 2 * t_2) * t_1 * y_s^2$	=	64,69 cm^4
	Iz=	317,22 cm^4

I_y =	$h^3 / 12 * t_1 + 2 * (b * t_2^3 / 12 + b * t_2 * ((h + t_2) / 2)^2)$	=	4683,02 cm^4
$W_{el,z}$ =	$I_z / (b + t_1 / 2 - y_s)$	=	36,99 cm³
$W_{el,y}$ =	$I_y / ((h + t_1) / 2)$	=	308,09 cm³

Effective web area:

Flangee:

As in Table 5.3.3:

ψ =		1,00	
k_σ =		0,43	

as in 5.3.5(3)

$\lambda_{trans,p}$ =	$b / t_2 / (28,4 * \varepsilon * \sqrt{(k_\sigma)})$	=	0,85 >0,673
ρ =	$(\lambda_{trans,p} - 0,22) / \lambda_{trans,p}^2$	=	0,872
b_{eff} =	$\rho * b$	=	8,28 cm
A_{eff} =	$2 * (b_{eff} * t_2) + (h + 2 * t_2) * t_1$	=	40,54 cm²

Location of shear centre:

$y_{s,eff}$ =	$(b_{eff} * (2 * t_2) * (b_{eff} + t_2) / 2) / A_{eff}$	=	1,088 cm
e_{Nz} =	$y_s - y_{s,eff}$	=	0,337 cm
β_A =	A_{eff} / A	=	0,965

Slenderness and reductions:

In Z-Z Axis:

$N_{cr,z}$ =	$\pi^2 * E * I_z / (H * 100)^2$	=	4109,22 kN
$\lambda_{trans,z}$ =	$\sqrt{(\beta_A * A * f_y / N_{cr,z})}$	=	1,52

Apply strut curve c

α =			0,49
φ =	$0,5 *(1 + \alpha * (\lambda_{trans,z} - 0,2) + \lambda_{trans,z}^2)$	=	1,979
χ_z =	$1 / (\varphi + (\varphi^2 - \lambda_{trans,z}^2)^{0,5})$	=	0,3080

In Y-Y Axis:

$N_{cr,y}$ =	$\pi^2 * E * I_y / (H * 100)^2$	=	60663,06 kN
$\lambda_{trans,y}$ =	$\sqrt{(\beta_A * A * f_y / N_{cr,y})}$	=	0,40

Apply strut curve c

α =			0,49
φ =	$0,5 * (1 + \alpha * (\lambda_{trans,y} - 0,2) + \lambda_{trans,y}^2)$	=	0,629
χ_y =	$1 / (\varphi + (\varphi^2 - \lambda_{trans,y}^2)^{0,5})$	=	0,8973
χ =	$MIN(\chi_z; \chi_y)$	=	0,3080
$N_{b,Rd}$ =	$\chi * A_{eff} * f_y / \gamma_M$	=	2667,53 kN

Limit bending moment about the Y-Y axis:

Cross-section class 4

$z'_s =$	$(-b_{eff} * t_2 * (h + t_2) / 2 + (b + (t_1 / 2)) * t_2 * (h + t_2) / 2) / A_{eff}$	=	0,382 cm
	$(h + t_2)^3 / 12 * t_1 + (h + t_2) * t_1 * z'_s$	=	2261,46 cm^4
	$b_{eff} * t_2 * ((h + t_2) / 2 + z'_s)^2 + (b + (t_1 / 2)) * t_2 * ((h + t_2) / 2 - z'_s)^2$	=	2457,57 cm^4
		Ieff,y=	4719,03 cm^4
$W_{eff,y} =$	$I_{eff,y} / ((h + t_2) / 2 + z'_s)$	=	306,79 cm^3
$M_{c,Rd,y} =$	$W_{eff,y} * f_y / \gamma_M$	=	65541,50 kNm

Limit bending moment about the Z-Z axis:

Cross-section class 4 Tab 5.3.3

$\psi =$	$-ys / (b + (t_1 / 2) - y_s)$	=	-0,17
$k_\sigma =$	$0,57 - 0,21 * \psi + 0,07 * \psi^2$	=	0,61
	$(b + (t_1 / 2)) / t_2 / (21 * \varepsilon * \sqrt{(k_\sigma)})$	=	1,02 > 1

As in Table:5.3.5 (3)

$\lambda_{trans,p} =$	$(b + (t_1 / 2)) / t_2 / (28,4 * \varepsilon * \sqrt{(k_\sigma)})$	=	0,75 > 0,673
$\rho =$	$(\lambda_{trans,p} - 0,22) / \lambda_{trans,p}^2$	=	0,942
$b_{eff} =$	$\rho * (b + (t_1 / 2)) / (1 - \psi)$	=	8,05 cm
$b_t =$	$-(b + (t_1 / 2)) * \psi / (1 - \psi)$	=	1,45 cm
$c_{eff} =$	$b_{eff} + b_t$	=	9,50 cm
$A_{eff} =$	$(h + t_2) * t_1 + 2 * c_{eff} * t_2$	=	41,40 cm^2
$y' =$	$2 * c_{eff}^2 * t_2 / 2 / A_{eff}$	=	1,31 cm
$I_{eff,z} =$	$(h + t_2) * t_1 * y'^2 + 2 * c_{eff}^3 / 12 * t_2 + 2 * c_{eff} * t_2 * (c_{eff} / 2 - y')^2$	=	272,12 cm^4
$W_{eff,z} =$	$I_{eff,z} / (c_{eff} / 2 - y')$	=	79,10 cm^3
$M_{c,Rd,z} =$	$W_{eff,z} * f_y / \gamma_M$	=	16898,64 kNm

Check buckling:

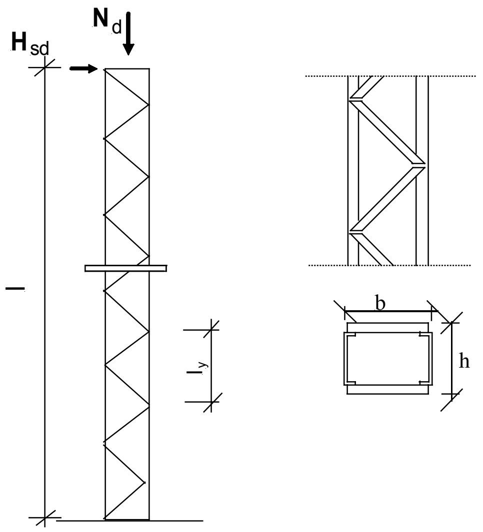
$N_{sd} =$	N_d	=	90,00 kN
$M_{y,sd} =$	$q_d * H^2 / 8$	=	30,00 kNm
$M_{z,sd} =$	$N_d * e_{Nz} / 100$	=	0,30 kNm

Hilfsbeiwerte:

$\beta_{My} =$			1,30
$\mu_y =$	$\lambda_{trans,y} * (2 * \beta_{My} - 4)$	=	-0,560 < 0,9
$k_y =$	$1 - (\mu_y * N_{sd}) / (\chi_y * A_{eff} * f_y)$	=	1,006 < 1,5
$\psi =$			1,000
$\beta_{M\psi} =$	$1,8 - 0,7 * \psi$	=	1,10 < 1,5
$\mu_z =$	$\lambda_{trans,z} * (2 * \beta_{M\psi} - 4)$	=	-2,736 < 0,9
$k_{z1} =$	$1 - (\mu_z * N_{sd}) / (\chi * A_{eff} * f_y)$	=	1,082
$k_{z2} =$			1,500
$k_z =$	$MIN(k_{z1}; k_{z2})$	=	1,082

$N_{sd}/(\chi*A_{eff}*f_y/\gamma_M)+k_y*100*M_{y,sd}/(W_{eff,y}*f_y/\gamma_M)+k_z*100*M_{z,sd}/(W_{eff,z}*f_y/\gamma_M)$ = **0,081 < 1**

Laced column made up of channel and angle sections:



Loads:

N_{sd} =			300,00 kN
H_{sd} =			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 φ_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_ζ =	TAB("steel/WG"; iζ; 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 =	$\sqrt{(235 / f_y)}$	=	1,00
λ_1 =	$93,90 * \varepsilon$	=	93,90

Partial safety factors:

γ_M = 1,10

Maximal tension force in chord:

Effective length ratio k = 2,00

Imperfection:

$w_o = 2 * l / 5 = 2{,}77$ cm

Effective moment of inertia:

$h_o = b - 2 * e_z = 34{,}60$ cm

$I_{eff} = 0{,}5 * h_o^2 * A_f = 35196{,}50\ cm^4$

Shear strength:

$S_v = 2 * E/10 * A_D * l_y * h_o^2 / (2 * \sqrt{(2 * h_o^2)^3}) = 71276{,}36$ kN

$N_{cr} = 1 / ((4 * (100 * l)^2/(\pi^2 * E/10 * I_{eff})) + (1 / S_v)) = 3615{,}26$ kN

$max\_M_s = H_{sd} * l + N_{sd} / (1 - N_{sd} / N_{cr}) * w_o / 100 = 320{,}46$ kNm

$N_{f,sd} = 0{,}5 * N_{sd} + 100 * max\_M_s / h_o = 1076{,}18$ kN

Analysis of channels:

$\lambda = l_y / i_z = 23{,}86$

$\lambda_{trans} = \lambda / (\lambda_1 * \varepsilon) = 0{,}254$

Apply strut curve c

$\alpha = 0{,}49$

$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans} - 0{,}2) + \lambda_{trans}^2) = 0{,}545$

$\chi = 1 / (\varphi + (\varphi^2 - \lambda_{trans}^2)^{0,5}) = 0{,}9735$

$N_{b,Rd} = \chi * A_f * f_y / 10 / \gamma_M = 1222{,}89$ kN

$N_{f,sd} / N_{b,Rd}$ = **0,88 < 1**

Analysis of lacing angles:

$max\_V_s = H_{sd} + N_{sd} / (1 - N_{sd} / N_{cr}) * w_o * \pi / (200 * l) = 47{,}06$ kN

Design force for a diagonal:

$N_{dS} = max\_V_s / (2 * COS(\varphi_1)) = 33{,}28$ kN

$\lambda = \sqrt{(2 * h_o^2)} / i_\zeta = 49{,}93$

$\lambda_{trans} = \lambda / (\lambda_1 * \varepsilon) = 0{,}532$

Apply strut curve c

$\alpha = 0{,}49$

$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans} - 0{,}2) + \lambda_{trans}^2) = 0{,}723$

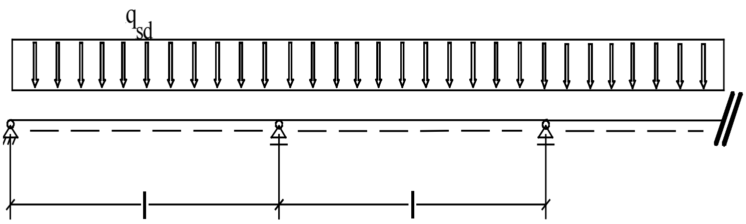
$\chi = 1 / (\varphi + (\varphi^2 - \lambda_{trans}^2)^{0,5}) = 0{,}8247$

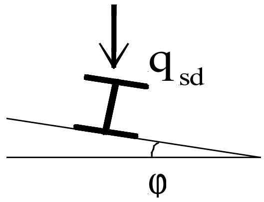
$N_{b,Rd} = \chi * A_D * f_y / 10 / \gamma_M = 84{,}57$ kN

$N_{dS}/N_{b,Rd}$ = **0,39 < 1**

Continuous Beam

Purlin / Strut with biaxial bending and torsional-flexural buckling:





Plan and elevation values:

Span length l=	720,00 cm
Angle of slope φ=	10,00 °
Spacing of purlin l'=	4,00 m
platischer Formbeiwert α_{ply} =	1,14
platischer Formbeiwert α_{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 cm^4
Moment of inertia I_z =	TAB("steel/"Typ; Iz; Name=Profil)	=	142,00 cm^4
Moment of inertia I_y =	TAB("steel/"Typ; IT; Name=Profil)	=	6,98 cm^4
I_ω =	$t_f * b_f^3 * (h - t_f)^2 / (24*10^6)$	=	12988,09 cm^6
i_z =	TAB("steel/"Typ; iz; Name=Profil)	=	2,24 cm
W_{ely}=	TAB("steel/"Typ; Wy; Name=Profil)	=	194,00 cm³
W_{ply} =	$\alpha_{ply} * W_{ely}$	=	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 =	$\sqrt{(235/f_y)}$	=	1,00

Partial safety factors:

γ_M =	1,10
γ_g =	1,35
γ_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_{bysd} =	$0{,}105 * q_{zd} * (l / 100)^2$	=	18,67 kNm
M_{bzsd} =	$0{,}105 * q_{yd} * (l / 100)^2$	=	3,27 kNm
V_{bzsd} =	$0{,}606 * q_{zd} * l / 100$	=	14,97 kN
V_{bysd} =	$0{,}606 * q_{yd} * l / 100$	=	2,62 kN

Check biaxial bending strength:

About Y-Y axis

A_v =	$1{,}04 * h * t_w / 10^2$	=	11,65 cm²
$V_{pl,z,Rd}$ =	$A_v * fy / (\gamma_M * \sqrt{(3)} * 10)$	=	143,69 kN
$V_{bzsd} / V_{pl,z,Rd}$		=	**0,10 < 1**
$V_{bzsd} / V_{pl,z,Rd}$		=	**0,10 < 0,5**

⇒ No reduction in moment strength necessary due to shear.

$M_{pl,y,Rd}$=	$W_{ply} * f_y / \gamma_M / 10$	=	4724,78 kNcm

About Z-Z axis

A_v =	$2 * b_f * t_f / 10^2$	=	17,00 cm²
$V_{pl,y,Rd}$ =	$A_v * fy / (\gamma_M * \sqrt{(3)} * 10)$	=	209,68 kN
$V_{bysd} / V_{pl,y,Rd}$		=	**0,01 < 1**
$V_{bysd} / V_{pl,y,Rd}$		=	**0,01 < 0,5**

⇒ No reduction in moment strength necessary due to shear

$M_{pl,z,Rd}$=	$1{,}5 * W_{elz} * f_y / \gamma_M / 10$	=	913,30 kNcm

Analysis:

as in 5.35: for I- and H- sections:

α= 2,00

β= 1,00

Or else: 5.4.8.1 (11)

$ABS((M_{bysd}/(M_{pl,y,Rd}/10^2))^{(\alpha)}+(M_{bzsd}/(M_{pl,z,Rd}/10^2))^{(\beta)})$ = **0,514 < 1**

Check flexural-torsional buckling strength:

$i_{LT} = (I_z * I_\omega / W_{ply}^2)^{0,25}$ = 2,48 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, Werte interpoliert.

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 =	$(h - t_f) / 10$	=	19,15 cm
5.5.2; Section class 1+2 β_w =			1,00
λ_1 =	$\pi * \sqrt{(E_s / f_y)}$	=	93,91
Auxilliary value v =$((k / k_w)^2+1/20*(k*l/i_{LT}/(h/t_f))^2+(2*C_2*z_g/h_s)^2)$			= 4,92
λ_{LT} =	$k*l/i_{LT}/((C_1)^{0,5}*(v^{0,5}-2*C_2*z_g/h_s)^{0,5})$	=	172,83
$\lambda_{trans,LT}$ =	$\lambda_{LT} / \lambda_1 * \sqrt{(\beta_w)}$	=	1,840

Apply strut curve a:

As in Table 5.5.1 α =	0,21		
φ =	$0,5 * (1 + \alpha * (\lambda_{trans,LT}-0,2) + \lambda_{trans,LT}^2)$	=	2,37
χ_{LT} =	$1 / (\varphi + (\varphi^2 - \lambda_{trans,LT}^2)^{0,5})$	=	0,26
No longitudinal forces k_z =			1,00
No longitudinal forces k_{LT} =			1,00

$k_{LT} * M_{bysd} / (\chi_{LT} * M_{pl,y,Rd} / 10^2) + k_z * M_{bzsd} / (M_{pl,z,Rd} / 10^2)$ = **<u>1,88 < 1</u>**

Section is not adequate: Restrain section in quarterly points:

l =	l / 4	=	180,00 cm
Auxilliary value v =$((k/k_w)^2+1/20*(k*l/i_{LT}/(h/t_f))^2+(2*C_2*z_g/h_s)^2)$			= 1,42
λ_{LT} =	$k*l/i_{LT}/((C_1)^{0,5}*(v^{0,5}-2*C_2*z_g/h_s)^{0,5})$	=	85,14
$\lambda_{trans,LT}$ =	$\lambda_{LT} / \lambda_1 * \sqrt{(\beta_w)}$	=	0,907

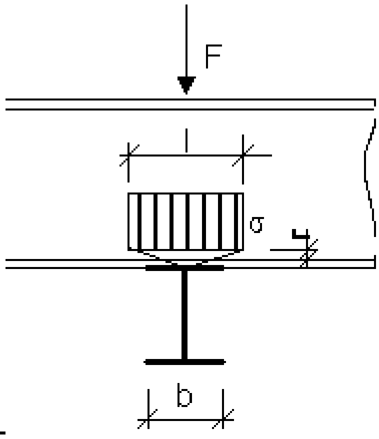
Apply strut curve a:

φ =	$0,5 * (1 + \alpha * (\lambda_{trans,LT}-0,2) + \lambda_{trans,LT}^2)$	=	0,99
χ_{LT} =	$1 / (\varphi + (\varphi^2 -\lambda_{trans,LT}^2)^{0,5})$	=	0,72

$k_{LT} * M_{bysd} / (\chi_{LT} * M_{pl,y,Rd} / 10^2) + k_z * M_{bzsd} / (M_{pl,z,Rd} / 10^2)$ = **<u>0,91 < 1</u>**

foot and support

Concentrated point load of a beam in another beam:



Design loads and properties

P_{sd} =			68,00 kN
M_{sd1}=			70,00 kNm
M_{sd2}=			-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
ε =	$\sqrt{(235 / f_y)}$	=	1,00
γ_M =			1,10
λ_1 =	93,90 * ε	=	93,90
f_{yd}=	f_y/γ_M	=	213,64 kN/cm²

Limit plastic bearing:

Stiff bearing length of top, bottom beam:

s_{s1} =	$s_o + 2 * t_o + 4 * r_o * (1 - \sqrt{(2)} / 2)$	=	31,84 mm
$\sigma_{f,Ed1}$ =	$100 * M_{sd1} / W_{elu}$	=	17,99 kN/cm²
b_{f1} =	$t_u * 25$	=	250,00 mm
b_{f1} =	$MIN(b_{f1}; b_u)$	=	200,00 mm
s_{y1} =	$2 * t_u * (b_u / s_u)^{0,5} * (1 - (\gamma_M * \sigma_{f,Ed1} / f_y * 10)^2)^{0,5}$	=	59,83 mm
$R_{y,Rd1}$ =	$(s_{s1} + s_{y1}) * s_u * f_y / \gamma_M / 10_3$	=	127,30 kN

$P_{sd} / R_{y,Rd1}$ = **0,53 < 1**

Stiff bearing length of bottom beam:

s_{s2} =	$s_u + 2 * t_u + 4 * r_u * (1 - \sqrt{(2)} / 2)$	=	47,59 mm
$\sigma_{f,Ed2}$ =	$100 * M_{sd2} / W_{elo}$	=	-15,07 kN/cm²
b_f =	$t_o * 25$	=	200,00 mm
b_{f2} =	$MIN(b_f; b_o)$	=	91,00 mm
s_{y2} =	$2 * t_o * (b_o / s_o)^{0,5} * (1 - (\gamma_M * \sigma_{f,Ed2} / f_y * 10)^2)^{0,5}$	=	46,99 mm
$R_{y,Rd2}$ =	$(s_{s2} + s_{y2}) * s_o * f_y / \gamma_M / 10^3$	=	107,09 kN

$P_{sd} / R_{y,Rd2}$ = **0,63 < 1**

Check web crushing:

Bottom beam:

m_1 =	s_{s1} / h_{1u}	=	0,238
m_2 =			0,200
m_u =	$MIN(m_1; m_2)$	=	0,200
$R_{a,Rd1}$ =	$0,5 * s_u^2 * (E_s * f_y)^{0,5} * ((t_u / s_u)^{0,5} + 3 *(s_u / t_u) * m_u) / \gamma_M / 10^3$	=	219,95 kN

$P_{sd} / R_{a,Rd1}$ = **0,31 < 1**

$M_{c,Rd1} = W_{plu} * f_y / \gamma_M / 10^3$ = 94,74 kNm

$M_{sd1} / M_{c,Rd1}$ = **0,74 < 1**

Top beam:

m_1 =	s_{s2} / h_{1o}	=	0,326
m_2 =			0,200
m_o =	$MIN(m_1; m_2)$	=	0,200
$R_{a,Rd2}$ =	$0,5 * s_o^2 * (E_s * f_y)^{0,5} * ((t_o / s_o)^{0,5} + 3 * (s_o / t_o) * m_o) / \gamma_M / 10^3$	=	145,85 kN

$P_{sd} / R_{a,Rd2}$ = **0,47 < 1**

$M_{c,Rd2} = W_{plo} * f_y / \gamma_M / 10^3$ = 35,56 kNm

$ABS(M_{sd2}) / M_{c,Rd2}$ = **0,62 < 1**

Check web buckling for whole web:

Bottom beam:

Effective web width b_{eff2} =	$(h_u^2 + s_{s1}^2)^{0,5}$	=	192,65 mm
Effective web area A_2 =	$b_{eff2} * s_u / 100$	=	12,52 cm²
Moment of inertia I_2 =	$b_{eff2} / 10 * (s_u / 10)^3 / 12$	=	0,441 cm⁴
Radius of gyration i_2 =	$(I_2 / A_2)^{0,5}$	=	0,188 cm
Effective slenderness λ_{trans2} =	$h_u / (i_2 * \lambda_1 * 10)$	=	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 =	$1 / (\varphi + (\varphi^2 - \lambda_{trans2}^2)^{0,5})$	=	0,4968
$R_{b,Rd2}$ =	$\chi_2 * A_2 * f_y / 10 / \gamma_M$	=	132,88 kN

$P_{sd} / R_{b,Rd2}$ = **0,51 < 1**

Top beam:

Effective web width b_{eff1} =	$(h_o^2 + s_{s1}^2)^{0,5}$	=	182,79 mm
Effective web area A_1 =	$b_{eff1} * s_o / 100$	=	9,69 cm²
Moment of inertia I_1 =	$b_{eff1} / 10 * (s_o / 10)^3 / 12$	=	0,227 cm⁴
Radius of gyration i_1 =	$(I_1 / A_1)^{0,5}$	=	0,153 cm
Effective slenderness λ_{trans1} =	$h_o / (i_1 * \lambda_1 * 10)$	=	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 =	$1 / (\varphi + (\varphi^2 - \lambda_{trans1}^2)^{0,5})$	=	0,4093
$R_{b,Rd1}$ =	$\chi_1 * A_1 * f_y / 10 / \gamma_M$	=	84,73 kN

$P_{sd} / R_{b,Rd1}$ = **0,80 < 1**

Check web yield at load points:

Bottom beam:

$\sigma_{x,Ed2}$ =	$M_{sd1} * 10^2 * (h_u / 2 - t_u) / I_{yu}$	=	161,25 kN/cm²
$\sigma_{z,Ed2}$ =	$10^3 * P_{sd} / ((s_{s1} + 2 * t_u) * s_u)$	=	201,80 kN/cm²

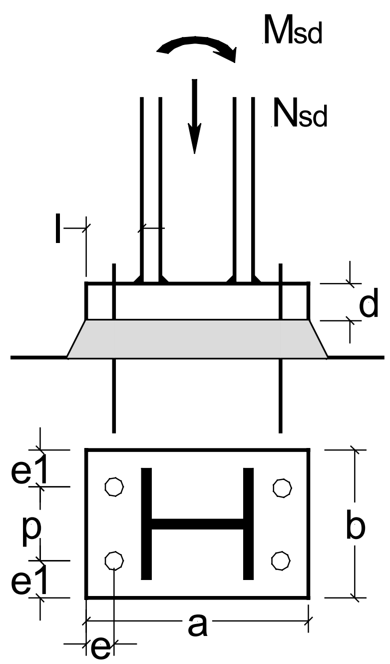
$(\sigma_{x,Ed2} / f_{yd})^2 + (\sigma_{z,Ed2} / f_{yd})^2 - (\sigma_{x,Ed2} / f_{yd})*(\sigma_{z,Ed2} / f_{yd})$ = **0,75 < 1**

Top beam:

$\sigma_{x,Ed1}$ =	$-M_{sd2} * 10^2 * (h_o / 2 - t_o) / I_{yo}$	=	136,67 kN/cm²
$\sigma_{z,Ed1}$ =	$10^3 * P_{sd} / ((s_{s2} + 2 * t_o) * s_o)$	=	201,76 kN/cm²

$(\sigma_{x,Ed1} / f_{yd})^2 + (\sigma_{z,Ed1} / f_{yd})^2 - (\sigma_{x,Ed1} / f_{yd}) * (\sigma_{z,Ed1} / f_{yd})$ = **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_{sd} =	1000,00 kN
M_{sd} =	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

γ_{Mb}= 1,25

γ_c= 1,50

γ_{M0}= 1,10

Design:

m =	$l - e - 0,8 * a_w * \sqrt{(2)}$	=	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} =	$MIN(\pi * 2 * m; \pi * m + 2 * e_1)$	=	274,51 mm
l_{eff2b} =	$MIN(\pi * m + p; p + 2 * e_1)$	=	437,26 mm
l_{eff} =	$MIN(l_{eff2a}; l_{eff2b}; l_{eff1})$	=	**215,51 mm**

Maximum stress of base plate at the centre of tension bolt:

$M_{pl,1,Rd}$ =	$0,25 * l_{eff} * (d / 10)^2 * f_y / 10 / \gamma_{M0}$	=	$12,12*10^3$ kNmm
n =	e	=	45,00 mm
n / (1,25 * m)		=	0,82 < 1
$B_{t,Rd}$ =	$0,9 * f_u * A_s / 10 / \gamma_{Mb}$	=	101,66 kN

Full flange yield:

$F_{T,Rd1}$ =	$4 * M_{pl,1,Rd} / m$	=	1109,64 kN

Bolt failure with flange yielding:

$F_{T,Rd2}$ =	$(2 * M_{pl,1,Rd} + n * 2 * B_{t,Rd}) / (m + n)$	=	376,47 kN

Bolt failure without flange yielding:

$F_{T,Rd3}$ =	$2 * B_{t,Rd}$	=	203,32 kN
$F_{T,Rd}$ =	$MIN(F_{T,Rd1}; F_{T,Rd2}; F_{T,Rd3})$	=	**203,32 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 =	$(a_1^2 / (a * b))^{0,5}$	=	3,80
f_j =	$\beta_j * k_j * f_{ck} / \gamma_c$	=	50,92 N/mm²
A_{eff} =	$10 * (N_{sd} + F_{T,Rd}) / f_j$	=	236,32 cm²
c =	$d * (f_y *10/ (3 * f_j * \gamma_{M0}))^{0,5}$	=	121,36 mm
x_0 =	$100 * A_{eff} / (2 * c + b_f)$	=	43,54 mm

$x_0 /(t_f + 2 * c)$ = **0,17 < 1**

Limit moment at column base:

$r_c =$ $h_t / 2 + c - x_0 / 2$ = 249,59 mm

$M_{Rd} =$ $(F_{T,Rd} * 10^3 *(h_t/2 + 100 - e) + A_{eff} * 100 * f_j * r_c) / 10^6$ = 342,02 kNm

Check permissible strength of column to bending and compression.:

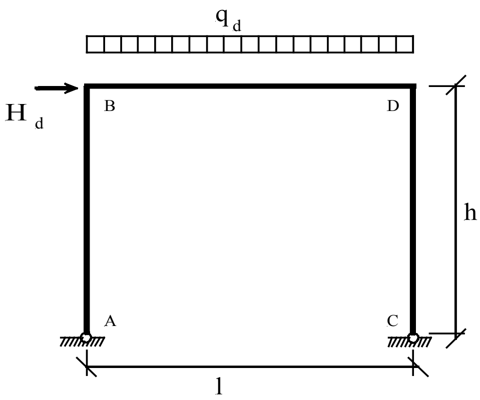
$M_{Ny,Rd} =$ $1,1 * M_{pl,y,Rd} * (1- N_{sd} / N_{pl,Rd})$ = 318,32 kNm

Analysis:

$M_{sd} / MIN(M_{Ny,Rd}; M_{Rd})$ = **0,70 < 1**

Frames

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 cm²
Moment of resistance W_{y1}=	TAB("steel/"Typ1; Wy; Name=Profil1)	=	324,00 cm²
Moment of resistance Wpl_{y1}=	1,14 * W_{y1}	=	369,36 cm³

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 $I_{\omega 2}$ = cm^6	10³*TAB("steel/"Typ2; Iω; Name=Profil2)	=486900,00	
Cross-sectional area A_2=	TAB("steel/"Typ2; A; Name=Profil2)	=	106,000 cm²
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 cm³
Moment of resistance W_{ply2} =	1,25 * W_{ely2}	=	1172,50 cm³

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
ε =	$\sqrt{(235 / f_y)}$	=	1,00
γ_M =			1,10
λ_1 =	93,90 * ε	=	93,90

Frame classification as in 5.2.5.2.:

Frame sways

k =	$I_{y1} / I_{y2} * h / l$	=	0,259

Deflection at top of frame due to horizontal load:

H =			1,00 kN
δ =	$(100 * h)^3 / (1,2 * E_s * I_{y2}) * (2 * k + 1) / k * H$	=	0,446 cm

Sumtotal of vertical loads:

V =	$q_d * l$	=	72,00 kN

Bracing is restrained out of plane of frame

δ / (h * 100) * V / H	=	0,054 < 0,1

⇒ Sway frame, plastic theory, restrained from moving out of plane.

Design with plastic theory

Imperfection:

φ_0 =	1 / 200	=	0,005
k_c =	$\mathrm{MIN}(\sqrt{(0,5 + 1 / n_c)}; 1)$	=	1,00
k_s =	$\mathrm{MIN}(\sqrt{(0,2 + 1 / n_s)}; 1)$	=	1,00
φ =	$\varphi_0 * k_c * k_s$	=	0,0050
ΔH_d =	$\varphi * V$	=	0,36 kN
H_{sd} =	$H_d + \Delta H_d$	=	9,36 kN

Design calculations:

Reactions at column bases:

H_l =	$q_d * l^2 / (4 * h * (2 * k + 3))$	=	6,82 kN
V_l =	$q_d * l / 2$	=	36,00 kN
M_{Bl} =	$-H_l * h$	=	-40,92 kNm
H_2 =	$H_{sd} / 2$	=	4,68 kN
V_2 =	$H_{sd} * h / l$	=	7,02 kN
M_{B2} =	$H_2 * h$	=	28,08 kNm

Analysis of column

N_{sd} =	$V_l + V_2$	=	43,02 kN
V_{sd} =	$H_l + H_2$	=	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:

$k_c =$ $I_{y2} / h / 100$ = 18,77 cm³

$k_1 =$ $1{,}5 * I_{y1} / l / 100$ = 7,29 cm³

$k_2 =$ 0,0

$\eta_2 =$ 1 (Hinge)

$\eta_1 =$ $k_c / (k_c + k_1 + k_2)$ = 0,72

Slenderness ratio as in Fig. E.2.1.:

$l =$ $3{,}2 * h$ = 19,20 m

$\lambda_y =$ $100 * l / i_y$ = 186,41

$\lambda_{trans,y} = \lambda_y / (\lambda_1 * \varepsilon)$ = 1,99

h_2 / b_2 = 1,00 < 1,2

Apply strut curve b:

$\alpha =$ 0,34

$\beta_A =$ 1,000

$\varphi =$ $0{,}5 * (1 + \alpha * (\lambda_{trans,y} - 0{,}2) + \lambda_{trans,y}^2)$ = 2,784

$\chi_y =$ $1 / (\varphi + (\varphi^2 - \lambda_{trans,y}^2)^{0,5})$ = 0,2114

Buckling outside the plane of frame

$\lambda_z =$ $100 * h / i_z$ = 98,68

$\lambda_{trans,z} = \lambda_z / (\lambda_1 * \varepsilon)$ = 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

$\chi_z =$ $1 / (\varphi + (\varphi^2 - \lambda_{trans,z}^2)^{0,5})$ = 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

$\mu_y =$ $\lambda_{trans,y} * (2 * \beta_{My} - 4) + (W_{ply2} - W_{ely2}) / W_{ely2}$ = -0,546 < 0,9

$k_y =$ $1 - (\mu_y * N_{sd}) / (\chi_y * A_2 * f_y)$ = 1,004 < 1,5

$N_{sd} / (\chi min * A_2 * f_y / 10 / \gamma_M) + 10^3 * k_y * ABS(M_D) / (W_{ply2} * f_y / \gamma_M)$ = <u>**0,37 < 1**</u>

Check torsional-flexural buckling:

$C_1 =$ 1,879

$M_{cr} = (C_1 * \pi^2 * E_s * I_{z2} / (h*100)^2 / 10 * (I_{\omega 2} / I_{z2} + ((h*100)^2 * G * I_{t2}) / (\pi^2 * E_s * I_{z2}))^{0,5})$ = 94241,31 kNcm

As in Table: F.1.1

$\psi =$ 0,00

$\beta_w =$ 1,00

$k =$ 1,00

$\lambda_{LT} =$ $\sqrt{(\pi^2 * E_s / 10 * W_{ply2} / M_{cr})}$ = 50,78

$\lambda_{trans,LT} = \lambda_{LT} / \lambda_1 * \sqrt{(\beta_w)}$ = 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

$\chi_{LT} =$ $1 / (\varphi + (\varphi^2 - \lambda_{trans,LT}^2)^{0,5})$ = 0,9114

$\beta_{M,LT} =$ $1,8 - 0,7 * \psi$ = 1,80

$\mu_{LT} =$ $0,15 * \lambda_{trans,z} * \beta_{M,LT} - 0,15$ = 0,134 < 0,9

$k_{LT} =$ $1 - (\mu_{LT} * N_{sd}) / (\chi_{LT} * A_2 * f_y)$ = 1,000 < 1,5

$N_{sd}/(\chi_z*A_2*f_y/10/\gamma_M)+10^3*k_{LT}*ABS(M_D)/(\chi_{LT}*W_{ply2}*f_y/\gamma_M)$ = **0,339 < 1**

Section analysis at D:

Column:

$A_v =$	$1,04 * h_2 * s_2 / 10^2$	=	24,96 cm²
$V_{pl,Rd} =$	$A_v * f_y / 10 / \sqrt{(3)} / \gamma_M$	=	307,86 kN
$V_{sd} / V_{pl,Rd}$	=	**0,037 < 0,5**	

No interaction

$N_{pl,Rd} =$	$A_2 * f_y / 10 / \gamma_M$	=	2264,55 kN
$n =$	$N_{sd} / N_{pl,Rd}$	=	0,02
$M_{pl,y,Rd} =$	$W_{ply2} * f_y / (\gamma_M * 10^3)$	=	250,49 kNm
$M_{Ny,Rd} =$	$1,11 * M_{pl,y,Rd} * (1 - n)$	=	272,48 kNm

$ABS(M_D) / MIN(M_{pl,y,Rd}; M_{Ny,Rd})$ = **0,28 < 1**

Beam:

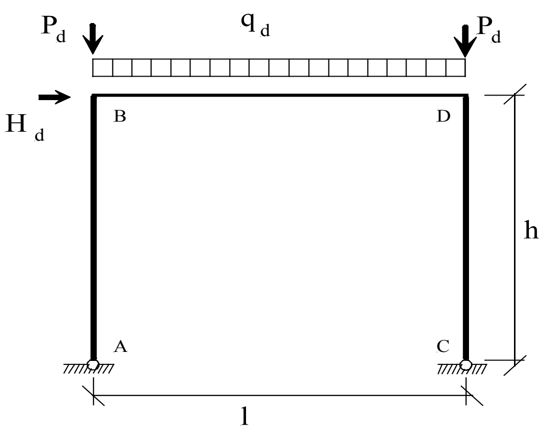
$A_v =$	$1,04 * h_1 * s_1 / 10^2$	=	15,48 cm²
$V_{pl,Rd} =$	$A_v * f_y / 10 / \sqrt{(3)} / \gamma_M$	=	190,93 kN
$N_{sd} / V_{pl,Rd}$	=	**0,225 < 0,5**	

No interaction

$N_{pl,Rd} =$	$A_1 * f_y / 10 / \gamma_M$	=	835,32 kN
$n =$	$V_{sd} / N_{pl,Rd}$	=	0,014
$M_{pl,y,Rd} =$	$W_{ply1} * f_y / (\gamma_M * 10^3)$	=	78,91 kNm
$M_{Ny,Rd} =$	$1,11 * M_{pl,y,Rd} * (1 - n)$	=	86,36 kNm

$ABS(M_D) / MIN(M_{pl,y,Rd}; M_{Ny,Rd})$ = **0,87 < 1**

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 cm²
Moment of resistance W_{y1}=	TAB("steel/"Typ1; Wy; Name=Profil1)	=	324,00 cm²
Moment of resistance Wpl_{y1}=	1,14 * W_{y1}	=	369,36 cm³

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 $I_{\omega 2}$ =	10³*TAB("steel/"Typ2; Iω; Name=Profil2)	=	486900,00 cm^6
Cross-sectional area A_2=	TAB("steel/"Typ2; A; Name=Profil2)	=	106,000 cm²
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 cm³
Moment of resistance W_{ply2} =	1,25 * W_{ely2}	=	1172,50 cm³

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
ε =	$\sqrt{(235 / f_y)}$	=	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 =	$I_{y1} / I_{y2} * h / l$	=	0,259

Deflection at top of frame due to horizontal load:

H =			1,00 kN
δ =	$(100 * h)^3 / (1,2 * E_s * I_{y2}) * (2 * k + 1) / k * H$	=	0,446 cm

Sumtotal of vertical loads:

V =	$P_d * 2 + q_d * l$	=	264,00 kN
$\delta / (h * 100) * V / H$		=	0,196 > 0,1

⇒ Frame, laterally unrestrained!;restrained from moving out of plane. mit Vergrößerungsfaktor!

Design with plastic theory and magnifying factor

Imperfection:

φ_0 =	1 / 200	=	0,005
k_c =	MIN($\sqrt{(0,5 + 1 / n_c)}$; 1)	=	1,00
k_s =	MIN($\sqrt{(0,2 + 1 / n_s)}$; 1)	=	1,00
φ =	$\varphi_0 * k_c * k_s$	=	0,0050
ΔH_d =	$\varphi * V$	=	1,32 kN
H_{sd} =	$H_d + \Delta H_d$	=	10,32 kN

Design calculations:

Reactions at column bases:

H_l =	$q_d * l^2 / (4 * h * (2 * k + 3))$	=	6,06 kN
V_l =	$q_d * l / 2$	=	32,00 kN
M_{Bl} =	$-H_l * h$	=	-36,36 kNm
H_2 =	$H_{sd} / 2$	=	5,16 kN
V_2 =	$H_{sd} * h / l$	=	7,74 kN
M_{B2} =	$H_2 * h$	=	30,96 kNm

Analysis of column

Dischinger factor:

D =	$1 / (1 - (\delta / (h * 100) * V / H))$	=	1,244
M_D =	$M_{Bl} - D * M_{B2}$	=	-74,87 kNm
N_{sd} =	$V_l + D * V_2 + P_d$	=	141,63 kN
V_{sd} =	$H_l + D * H_2$	=	12,48 kN

Buckling in the plane of frame:

k_c =	$I_{y2} / h / 100$	=	18,77 cm³
k_1 =	$I_{y1} / l / 100$	=	4,86 cm³
k_2 =			0,0
η_2 =			1 (Hinge)
η_1 =	$k_c / (k_c + k_1 + k_2)$	=	0,79

Slenderness ratio as in Fig. E.2.1.:

$l = 0{,}92 * h = 5{,}52$ m

$\lambda_y = 100 * l / i_y = 53{,}59$

$\lambda_{trans,y} = \lambda_y / (\lambda_1 * \varepsilon) = 0{,}57$

$h_2 / b_2 = 1{,}00 < 1{,}2$

Apply strut curve b:

$\alpha = 0{,}34$

$\beta_A = 1{,}000$

$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans,y} - 0{,}2) + \lambda_{trans,y}^2) = 0{,}725$

$\chi_y = 1 / (\varphi + (\varphi^2 - \lambda_{trans,y}^2)^{0,5}) = 0{,}8525$

Buckling outside the plane of frame

$\lambda_z = 100 * h / i_z = 98{,}68$

$\lambda_{trans,z} = \lambda_z / (\lambda_1 * \varepsilon) = 1{,}05$

Apply strut curve c:

$\alpha = 0{,}49$

$\beta_A = 1{,}000$

$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans,z} - 0{,}2) + \lambda_{trans,z}^2) = 1{,}260$

$\chi_z = 1 / (\varphi + (\varphi^2 - \lambda_{trans,z}^2)^{0,5}) = 0{,}5111$

$\chi_{min} = \mathrm{MIN}(\chi_y; \chi_z) = 0{,}5111$

Check bending and buckling:

$\psi = 0{,}00$

$\beta_{My} = 1{,}8 - 0{,}7 * \psi = 1{,}80$

$\mu_y = \lambda_{trans,y} * (2 * \beta_{My} - 4) + (W_{ply2} - W_{ely2}) / W_{ely2} = 0{,}022 < 0{,}9$

$k_y = 1 - (\mu_y * N_{sd}) / (\chi_y * A_2 * f_y) = 1{,}000 < 1{,}5$

$N_{sd} / (\chi min*A_2*f_y/10/\gamma_M) + 10^3*k_y*ABS(M_D) / (W_{ply2}*f_y/\gamma_M) =$ **0,42 < 1**

Check torsional-flexural buckling:

$C_1 = 1{,}879$

$M_{cr} = (C_1*\pi^2*E_s*I_{z2}/(h*100)^2/10*(I_{\omega 2}/I_{z2}+((h*100)^2*G*I_{t2})/(\pi^2*E_s*I_{z2}))^{0,5}) = 94241{,}31$ kNcm

As in Table: F.1.1

$\psi = 0{,}00$

$\beta_w = 1{,}00$

$k = 1{,}00$

$\lambda_{LT} = \sqrt{(\pi^2 * E_s / 10 * W_{ply2} / M_{cr})} = 50{,}78$

$\lambda_{trans,LT} = \lambda_{LT} / \lambda_1 * \sqrt{(\beta_w)} = 0{,}541$

Apply strut curve a

$\alpha = 0{,}21$

$\varphi = 0{,}5 * (1 + \alpha * (\lambda_{trans,LT} - 0{,}2) + \lambda_{trans,LT}^2) = 0{,}682$

$\chi_{LT} = 1 / (\varphi + (\varphi^2 - \lambda_{trans,LT}^2)^{0,5}) = 0{,}9114$

$\beta_{M,LT} = 1{,}8 - 0{,}7 * \psi = 1{,}80$

$\mu_{LT} = 0{,}15 * \lambda_{trans,z} * \beta_{M,LT} - 0{,}15 = 0{,}134 < 0{,}9$

$k_{LT} = 1 - (\mu_{LT} * N_{sd}) / (\chi_{LT} * A_2 * f_y) = 0{,}999 < 1{,}5$

$N_{sd} / (\chi_z*A_2*f_y/10/\gamma_M) + 10^3*k_{LT}*ABS(M_D) / (\chi_{LT}*W_{ply2}*f_y/\gamma_M) =$ **0,450 < 1**

Section analysis at D:
Column:

A_v =	$1{,}04 * h_2 * s_2 / 10^2$	=	24,96 cm²
$V_{pl,Rd}$ =	$A_v * f_y / 10 / \sqrt{(3)} / \gamma_M$	=	307,86 kN
$V_{sd} / V_{pl,Rd}$	=	**0,041 < 0,5**	

No interaction

$N_{pl,Rd}$ =	$A_2 * f_y / 10 / \gamma_M$	=	2264,55 kN
n =	$N_{sd} / N_{pl,Rd}$	=	0,063
$M_{pl,y,Rd}$ =	$W_{ply2} * f_y / (\gamma_M * 10^3)$	=	250,49 kNm
$M_{Ny,Rd}$ =	$1{,}11 * M_{pl,y,Rd} * (1 - n)$	=	260,53 kNm
$ABS(M_D) / MIN(M_{pl,y,Rd;}\ M_{Ny,Rd)}$	=	**0,30 < 1**	

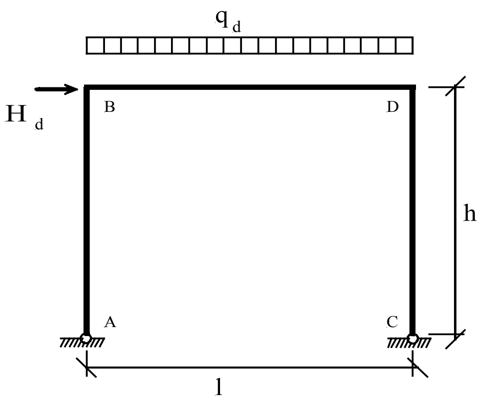
Beam:

A_v =	$1{,}04 * h_1 * s_1 / 10^2$	=	15,48 cm²
$V_{pl,Rd}$ =	$A_v * f_y / 10 / \sqrt{(3)} / \gamma_M$	=	190,93 kN
$N_{sd} / V_{pl,Rd}$	=	**0,742 < 0,5**	

No interaction

$N_{pl,Rd}$ =	$A_1 * f_y / 10 / \gamma_M$	=	835,32 kN
n =	$V_{sd} / N_{pl,Rd}$	=	0,015
$M_{pl,y,Rd}$ =	$W_{ply1} * f_y / (\gamma_M * 10^3)$	=	78,91 kNm
$M_{Ny,Rd}$ =	$1{,}11 * M_{pl,y,Rd} * (1 - n)$	=	86,28 kNm
$ABS(M_D) / MIN(M_{pl,y,Rd;}\ M_{Ny,Rd})$	=	**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 cm²
Moment of resistance W_{y1}=	TAB("steel/"Typ1; Wy; Name=Profil1)	=	324,00 cm²
Moment of resistance Wpl_{y1}=	1,14 * W_{y1}	=	369,36 cm³

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 $I_{\omega 2}$ = cm^6	10³*TAB("steel/"Typ2; Iω; Name=Profil2)	=486900,00	
Cross-sectional area A_2=	TAB("steel/"Typ2; A; Name=Profil2)	=	106,000 cm²
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 cm³
Moment of resistance W_{ply2} =	1,25 * W_{ely2}	=	1172,50 cm³

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
ε =	$\sqrt{(235 / f_y)}$	=	1,00
γ_M =			1,10
λ_1 =	$93,90 * \varepsilon$	=	93,90

Frame classification as in 5.2.5.2.:

Frame sways

k =	$I_{y1} / I_{y2} * h / l$	=	0,259

Deflection at top of frame due to horizontal load:

H =			1,00 kN
δ =	$(100*h)^3 *10 / (12*E_s*I_{y2}) * (2*k+1) / k*H$	=	0,446 cm

Sumtotal of vertical loads:

V =	$q_d * l$	=	72,00 kN

Bracing is restrained out of plane of frame

	$\delta / (h * 100) * V / H$	=	0,0535 < 0,1

⇒ Sway frame, plastic theory;restrained from moving out of plane.

Design with plastic theory and magnifying factor

Imperfection:

φ_0 =	1 / 200	=	0,005
k_c =	$MIN(\sqrt{(0,5 + 1 / n_c)}; 1)$	=	1,00
k_s =	$MIN(\sqrt{(0,2 + 1 / n_s)}; 1)$	=	1,00
φ =	$\varphi_0 * k_c * k_s$	=	0,0050
ΔH_d =	$\varphi * V$	=	0,36 kN
H_{sd} =	$H_d + \Delta H_d$	=	9,36 kN

Design calculations:

Reactions at column bases:

H_l =	$q_d * l^2 / (4 * h * (2 * k + 3))$	=	6,82 kN
V_l =	$q_d * l / 2$	=	36,00 kN
M_{Bl} =	$-H_l * h$	=	-40,92 kNm
H_2 =	$H_{sd} / 2$	=	4,68 kN
V_2 =	$H_{sd} * h / l$	=	7,02 kN
M_{B2} =	$H_2 * h$	=	28,08 kNm

Analysis of column

N_{sd} =	$V_l + V_2$	=	43,02 kN
V_{sd} =	$H_l + H_2$	=	11,50 kN

Dischinger factor:

D =	$1 / (1 - (\delta / (h * 100) * V / H))$	=	1,057
M_D =	$M_{Bl} - D * M_{B2}$	=	-70,60 kNm

Buckling in the plane of frame:

k_c =	$I_{y2} / h / 100$	=	18,77 cm³
k_1 =	$I_{y1} / l / 100$	=	4,86 cm³
k_2 =			0,0
η_2 =			1 (Hinge)
η_1 =	$k_c / (k_c + k_1 + k_2)$	=	0,79

Slenderness ratio as in Fig. E.2.1.:

l =	$0{,}92 * h$	=	5,52 m
λ_y =	$100 * l / i_y$	=	53,59
$\lambda_{trans,y}$ =	$\lambda_y / (\lambda_1 * \varepsilon)$	=	0,57
h_2 / b_2		=	1,00 < 1,2

Apply strut curve b:

α =			0,34
β_A =			1,000
φ=	$0{,}5*(1+\alpha*(\lambda_{trans,y}-0{,}2)+\lambda_{trans,y}^2)$	=	0,725
χ_y=	$1/(\varphi+(\varphi^2-\lambda_{trans,y}^2)^{0,5})$	=	0,8525

Buckling outside the plane of frame

λ_z=	$100*h/i_z$	=	98,68
$\lambda_{trans,z}$=	$\lambda_z/(\lambda_1*\varepsilon)$	=	1,05

Apply strut curve c:

α=			0,49
β_A =			1,000
φ =	$0{,}5 * (1 + \alpha * (\lambda_{trans,z} - 0{,}2) + \lambda_{trans,z}^2)$	=	1,260
χ_z =	$1 / (\varphi + (\varphi^2 - \lambda_{trans,z}^2)^{0,5})$	=	0,5111
χ_{min} =	$MIN(\chi_y; \chi_z)$	=	0,5111

Check bending and buckling:

ψ =	0,00		
β_{My} =	$1{,}8 - 0{,}7 * \psi$	=	1,80
μ_y =	$\lambda_{trans,y} * (2 * \beta_{My} - 4) + (W_{ply2} - W_{ely2}) / W_{ely2}$	=	0,022 < 0,9
k_y =	$1 - (\mu_y * N_{sd}) / (\chi_y * A_2 * f_y)$	=	1,000 < 1,5

$N_{sd} / (\chi_{min} * A_2 * f_y / 10 / \gamma_M) + 100 * k_y * ABS(M_D) / (W_{ply2} * f_y / 10 / \gamma_M)$ = <u>**0,32 < 1**</u>

Check torsional-flexural buckling:

C_1 = 1,879

M_{cr} = $(C_1*\pi^2*E_s*I_{z2} / (h*100)^2/10*(I_{\omega 2}/I_{z2}+((h*100)^2*G*I_{t2})/(\pi^2*E_s*I_{z2}))^{0,5})$ = 94241,31 kNcm

As in Table: F.1.1

ψ =	0,00		
β_w =	1,00		
k =	1,00		
λ_{LT} =	$\sqrt{}(\pi^2 * E_s / 10 * W_{ply2} / M_{cr})$	=	50,78
$\lambda_{trans,LT}$ =	$\lambda_{LT} / \lambda_1 * \sqrt{}(\beta_w)$	=	0,541

Apply strut curve a

α =			0,21
φ =	$0{,}5 * (1 + \alpha * (\lambda_{trans,LT} - 0{,}2) + \lambda_{trans,LT}^2)$	=	0,682
χ_{LT} =	$1 / (\varphi + (\varphi^2 - \lambda_{trans,LT}^2)^{0,5})$	=	0,9114
$\beta_{M,LT}$ =	$1{,}8 - 0{,}7 * \psi$	=	1,80
μ_{LT} =	$0{,}15 * \lambda_{trans,z} * \beta_{M,LT} - 0{,}15$	=	0,134 < 0,9
k_{LT} =	$1 - (\mu_{LT} * N_{sd}) / (\chi_{LT} * A_2 * f_y)$	=	1,000 < 1,5

$N_{sd} / (\chi_z * A_2 * f_y/10 / \gamma_M) + 100 * k_{LT} * ABS(M_D) / (\chi_{LT} * W_{ply2} * f_y/10 / \gamma_M)$ = <u>**0,346 < 1**</u>

Section analysis at D:

Column:

$A_v =$	$1{,}04 * h_2 * s_2 / 10^2$	=	24,96 cm²
$V_{pl,Rd} =$	$A_v * f_y / 10 / \sqrt{(3)} / \gamma_M$	=	307,86 kN
$V_{sd} / V_{pl,Rd}$	=	**0,037 < 0,5**	

No interaction

$N_{pl,Rd} =$	$A_2 * f_y / 10 / \gamma_M$	=	2264,55 kN
$n =$	$N_{sd} / N_{pl,Rd}$	=	0,02
$M_{pl,y,Rd} =$	$W_{ply2} * f_y / (\gamma_M * 10^3)$	=	250,49 kNm
$M_{Ny,Rd} =$	$1{,}11 * M_{pl,y,Rd} * (1 - n)$	=	272,48 kNm

$ABS(M_D) / MIN(M_{pl,y,Rd;}\ M_{Ny,Rd})$ = **0,28 < 1**

Beam:

$A_v =$	$1{,}04 * h_1 * s_1 / 10^2$	=	15,48 cm²
$V_{pl,Rd} =$	$A_v * f_y / 10 / \sqrt{(3)} / \gamma_M$	=	190,93 kN
$N_{sd} / V_{pl,Rd}$	=	**0,225 < 0,5**	

No interaction

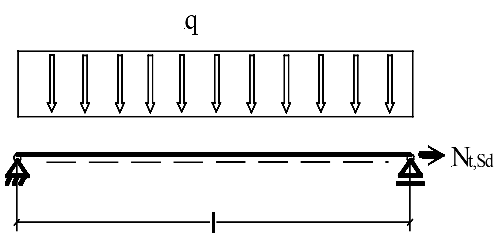
$N_{pl,Rd} =$	$A_1 * f_y / 10 / \gamma_M$	=	835,32 kN
$n =$	$V_{sd} / N_{pl,Rd}$	=	0,014
$M_{pl,y,Rd} =$	$W_{ply1} * f_y / (\gamma_M * 10^3)$	=	78,91 kNm
$M_{Ny,Rd} =$	$1{,}11 * M_{pl,y,Rd} * (1 - n)$	=	86,36 kNm

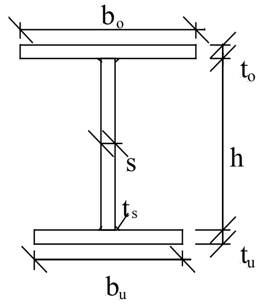
$ABS(M_D) / MIN(M_{pl,y,Rd;}\ M_{Ny,Rd})$ = **0,89 < 1**

Single-span beam

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 =	$\sqrt{(235/f_y)}$	=	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 =	$(A / 2 - (b_o * t_o)) / s$	=	58,75 cm
x_s =	h - x	=	41,25 cm

Flange:

c=	$((b_o - s) / 2 - t_s * \sqrt{(2)}) / t_o$	=	5,02 cm
$c / (9 * \varepsilon)$		=	0,69 < 1

⇒ Section class 1

Web:

α =	x / h	=	0,59 cm
$h / s / (456 * \varepsilon / (13 * \alpha - 1))$		=	0,90 < 1
$h / s / (396 * \varepsilon / (13 * \alpha - 1))$		=	1,04 > 1

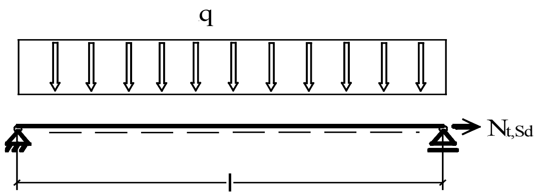
⇒ Section class 2

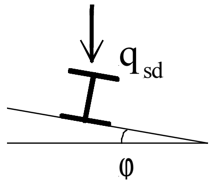
Check bending moment strength:

M_{sd}=	$\gamma_g * q * l^2 / 8$	=	392,69 kNm
Critical torsional-buckling moment:			
I_z=	$t_o * b_o^3 / 12 + t_u * b_u^3 / 12$	=	55125,00 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=	$(b_u * t_u * (h + t_o) + h * s * (h + t_o) / 2) / A$	=	55,27 cm
Location of shear centre:			
s_s=	$(h + (t_o + t_u) / 2) * (b_u^3 / (b_o^3 + b_u^3))$	=	68,45 cm
z_s=	$-(s_s - e)$	=	-13,18 cm
I_t=	$1/3 * (b_u * t_u^3 + b_o * t_o^3 + h * s^3)$	=	1552,92 cm^4
z_a=			0,00 cm
z_g=	$z_a - z_s$	=	13,18 cm
as in F 1.4:			
β_f=	$b_o^3 / (b_o^3 + b_u^3)$	=	0,339 < 0,5
z_j=	$(2 * \beta_f - 1) * (h + (t_o + t_u) / 2) / 2$	=	-16,66 cm
I_w=	$\beta_f * (1 - \beta_f) * I_z * (h + (t_o + t_u) / 2)^2$	=	$132,32*10^6$ cm^6
P_1=	$C_1 * (\pi^2 * E_s * I_z / (k * l * 100)^2)$	=	22453,85
P_2=	$(k/k_w)^2*(I_w/I_z+((k*l*100)^2*G*I_t)/(\pi^2*E_s*I_z)+(C_2*z_g-C_3*z_j)^2)^{0,5}$	=	94,66
P_3=	$C_2 * z_g - C_3 * z_j$	=	14,80
M_{cr}=	$P_1 * (P_2 - P_3) / 10^3$	=	1793,16 kNm
W_{pl} =	$b_u*t_u*(x_s+t_u/2)+x_s^2*s/2+b_o*t_o*(x+t_o/2)+x^2*s/2$	=	21148,13 cm^3
Section class 2 β_w=			1,00
$\lambda_{trans,LT}$=	$(\beta_w * W_{pl} * f_y / (10^3 * M_{cr}))^{0,5}$	=	2,046
As in Table 5.5.1 α=			0,49
φ=	$0,5 * (1 + \alpha * (\lambda_{trans,LT} - 0,2) + \lambda_{trans,LT}^2)$	=	3,05
χ_{LT}=	$1 / (\varphi + (\varphi^2 - \lambda_{trans,LT}^2)^{0,5})$	=	0,1883
$M_{b,Rd}$=	$\beta_w*\chi_{LT}*W_{pl}*f_y/10/(\gamma_M*10)^2$	=	1168,33 kNm

$M_{sd}/M_{b,Rd}$ = **0,34 < 1**

Pos.: Single-span beam with biaxial bending:





Span length l = 5,80 m
Angle of slope φ = 8,53 °

Loads:

q_{sd}= 7,00 kN/m
$N_{t,sd}$= 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=	$\sqrt{(235/f_y)}$	=	1,00

platischer Formbeiwert α_{ply} = 1,14
platischer Formbeiwert α_{plz} = 1,25
γ_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 cm^4
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} =	$\alpha_{ply} * W_{ely}$	=	176,70 cm³
W_{elz}=	TAB("steel/"Typ; Wz; Name=Profil)	=	55,60 cm³
W_{plz} =	$\alpha_{plz} * W_{elz}$	=	69,50 cm³

Design loads:

Plastic limit force:

$N_{pl,Rd}$= $A * fy / \gamma_M / 10$ = 670,82 kN

Limit force that plasticizes the web

$N_{pl,w,Rd}$= $(h - 2 * t) * s * f_y / \gamma_M / 10^3$ = 136,30 kN

$N_{t,sd} / MIN(0,5 * N_{pl,w,Rd}; N_{pl,Rd} / 4)$ = **1,47 > 1**

$\Rightarrow$ Reduce plastic limit force as in 5.4.8.1:

α =	$(A - 2 * b * t / 100) / A$	=	0,242 < 0,5
η =	$N_{t,sd} / N_{pl,Rd}$	=	0,149
$M_{Ny,Rd}$=	$((W_{ply} * f_y) / \gamma_M * (1 - \eta) / (1 - 0,5 * \alpha)) / 10^3$	=	36,55 kNm
η / α		=	0,62 < 1
$\Rightarrow M_{Nz,Rd}$=	$W_{plz} * f_y / (\gamma_M * 10^3)$	=	14,85 kNm

<u>Design momente:</u>

q_{sdy} =	q_{sd} * COS(φ)	=	6,92 kN/m
q_{sdz} =	q_{sd} * SIN(φ)	=	1,04 kN/m
$M_{y,sd}$ =	q_{sdy} * l² / 8	=	29,10 kNm
$M_{z,sd}$ =	q_{sdz} * l² / 8	=	4,37 kNm

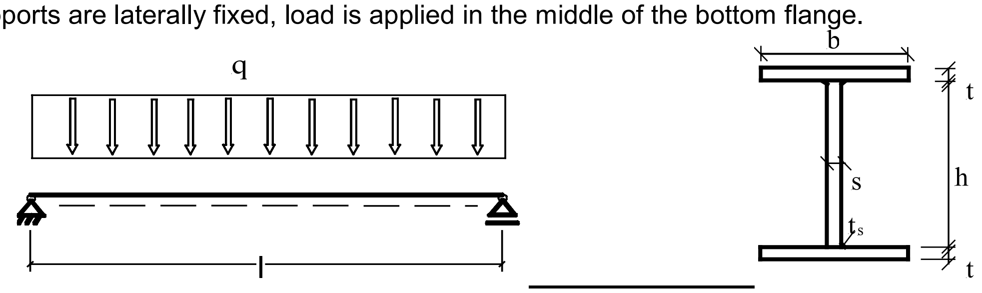
<u>Check section strength:</u>

for I-Profile gilt α=			2,00
β_1=	5 * η	=	0,75 < 1
β_2=			1,00
β =	MAX(β_1;β_2)	=	1,00

$(M_{y,sd} / M_{Ny,Rd})^{(\alpha)} + (M_{z,sd} / M_{Nz,Rd})^{\beta}$ = **<u>0,93 < 1</u>**

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_{sd} = 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
ε =	$\sqrt{(235/f_y)}$	=	0,81

Partial safety factors:

γ_M= 1,10

Section classification:

Flange:

c= $(b/2-t_s*\sqrt{(2)})/t$ = 3,72 cm

$c/(9*\varepsilon)$ = 0,51 < 1

⇒ Section class 1

Web:

d= $h-2*t_s*\sqrt{(2)}$ = 48,59 cm

$d/s/(72*\varepsilon)$ = 0,69 < 1

⇒ Section class 1

Check bending moment strength:

W_{pl} =	(b * t * (h + t) / 2 + (h / 2) * s * (h / 4)) * 2	=	3375,00 cm³
$M_{c,Rd}$=	$(W_{pl} * f_y / \gamma_M) / 10^3$	=	1089,20 kNm
M_{sd}=	q_{sd} * l² / 8	=	400,00 kNm

$M_{sd} / M_{c,Rd}$ = **<u>0,37 < 1</u>**

Check shear strength:

A_v=	h * s	=	60,00 cm²
$V_{pl,Rd}$=	A_v * (f_y / √(3)) / γ_M / 10	=	1117,96 kN
V_{sd}=	q_{sd} * l / 2	=	200,00 kN

V_{sd} / $V_{pl,Rd}$ = **0,18 < 1**

Check torsional buckling strength:

I_z=	2 * t * b³ / 12	=	3333,33 cm^4
I_w=	I_z * (h + t)² / 4	=	2,297*10^6 cm^6
I_t=	1/3 * (2 * b * t³ + h * s³)	=	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 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*(π²*E_s*I_z/(k*l*10³)²)/10	=	12,20
P_2=	$(k/k_w)^2 \cdot (I_w/I_z+((k \cdot l \cdot 100)^2 \cdot G \cdot I_t)/(\pi^2 \cdot E_s \cdot I_z)+(C_2 \cdot z_g-C_3 \cdot z_j)^2)^{0,5}$	=	51,27
P_3=	C_2*z_g-C_3*z_j	=	-12,65
M_{cr}=	P_1*(P_2-P_3)	=	779,82 kNm

Cross-section class 1 β_w= 1,00

$\lambda_{trans,LT}$= $(\beta_w \cdot W_{pl} \cdot f_y/M_{cr}/10^3)^{0,5}$ = 1,240

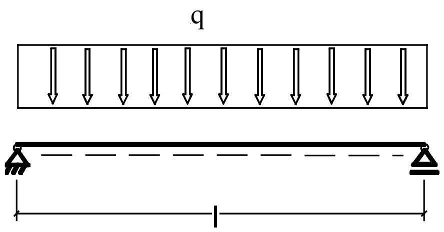
Apply strut curve c:

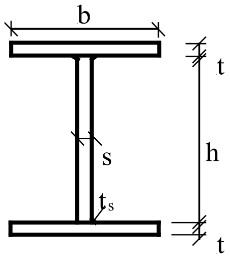
α_{LT}=			0,49
φ_{LT}=	$0,5 \cdot (1+\alpha_{LT} \cdot (\lambda_{trans,LT}-0,2)+\lambda_{trans,LT}^2)$	=	1,524
χ_{LT}=	$1/(\varphi_{LT}+(\varphi_{LT}^2-\lambda_{trans,LT}^2)^{0,5})$	=	0,415
$M_{b,Rd}$=	$\beta_w \cdot \chi_{LT} \cdot W_{pl} \cdot f_y/\gamma_M/10$	=	45201,99 kNcm

M_{sd}*100/$M_{b,Rd}$ = **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_{sd} = 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
ε =	$\sqrt{(235/f_y)}$	=	0,81

Partial safety factors:

γ_M= 1,10

Section classification As in Table 5.3.1:

Flange:

c=	$(b/2-t_s*\sqrt{(2)})/t$	=	3,72 cm
$c/(9*\varepsilon)$		=	0,51 < 1

Section class 1

Web:

d=	$h-2*t_s*\sqrt{(2)}$	=	48,59 cm
$d/s/(72*\varepsilon)$		=	0,69 < 1

Section class 1

Check bending moment strength:

W_{pl} =	(b*t*(h+t)/2+(h/2)*s*(h/4))*2	=	3375,00 cm³
$M_{c,Rd}$=	$(W_{pl}*f_y/\gamma_M)/10^3$	=	1089,20 kNm
M_{sd}=	$q_{sd}*l^2/8$	=	360,00 kNm

$M_{sd}/M_{c,Rd}$ = **0,33 < 1**

Check shear strength:

A_V=	h*s	=	60,00 cm²
$V_{pl,Rd}$=	$A_V*(f_y/\sqrt{(3)})/\gamma_M/10$	=	1117,96 kN
V_{sd}=	$q_{sd}*l/2$	=	180,00 kN

$V_{sd}/V_{pl,Rd}$ = **0,16 < 1**

Check torsional buckling strength:

I_z=	2 * t * b³ / 12	=	3333,33 cm^4
I_w=	I_z * (h + t)² / 4	=	$2{,}297*10^6$ cm^6
I_t=	1/3 * (2 * b * t³ + h * s³)	=	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

$M_{cr}= C_1*(\pi^2*E_s*I_z/(k*l*10^2)^2)*((k/k_w)^2*I_w/I_z+((k*l*10^2)^2*G*I_t)/(\pi^2*E_s*I_z))^{0,5}/10^3$ = 606,05 kNm

Cross-section class 1 β_w= 1,00

$\lambda_{trans,LT}= (\beta_w*W_{pl}*f_y/M_{cr}/10^3)^{0,5}$ = 1,406

Apply strut curve c:

α_{LT}= 0,49

$\varphi_{LT}= 0,5*(1+\alpha_{LT}*(\lambda_{trans,LT}-0,2)+\lambda_{trans,LT}^2)$ = 1,784

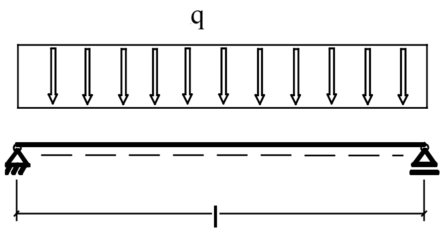
$\chi_{LT}= 1/(\varphi_{LT}+(\varphi_{LT}^2-\lambda_{trans,LT}^2)^{0,5})$ = 0,347

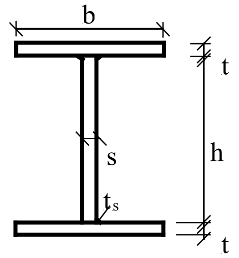
$M_{b,Rd}= \beta_w*\chi_{LT}*W_{pl}*f_y/\gamma_M/10^3$ = 377,95 kNm

$M_{sd} / M_{b,Rd}$ = **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_{sd} =	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
ε =	$\sqrt{(235/f_y)}$	=	0,81

Partial safety factors:

γ_M= 1,10

Section classification As in Table 5.3.1:

Flange:

c=	$(b/2-t_s*\sqrt{(2)})/t$	=	3,72 cm
$c/(9*\varepsilon)$		=	0,51 < 1

Section class 1

Web:

d=	$h-2*t_s*\sqrt{(2)}$	=	48,59 cm
$d/s/(72*\varepsilon)$		=	0,69 < 1

Section class 1

Check bending moment strength:

W_{pl} =	(b*t*(h+t)/2+(h/2)*s*(h/4))*2	=	3375,00 cm³
$M_{c,Rd}$=	$(W_{pl}*f_y/\gamma_M)/10^3$	=	1089,20 kNm
M_{sd}=	$q_{sd}*l^2/8$	=	288,00 kNm

$M_{sd}/M_{c,Rd}$ = **0,26 < 1**

Check shear strength:

$A_v=$	$h*s$	=	60,00 cm²
$V_{pl,Rd}=$	$A_v*(f_y/\sqrt{(3)})/\gamma_M/10$	=	1117,96 kN
$V_{sd}=$	$q_{sd}*l/2$	=	144,00 kN

$V_{sd}/V_{pl,Rd}$ = **0,13 < 1**

Check torsional buckling strength:

$I_z=$	$2 * t * b^3 / 12$	=	3333,33 cm^4
$I_w=$	$I_z * (h + t)^2 / 4$	=	$2{,}297*10^6$ cm^6
$I_t=$	$1/3 * (2 * b * t^3 + h * s^3)$	=	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

$z_g= \quad z_a - z_s \quad = \quad 27{,}50$ cm

Critical torsional-buckling moment

$P_1=$	$C_1*(\pi^2*E_s*I_z/(k*l*100)^2)$	=	12198,18 < 1
$P_2=$	$(k/k_w)^2*(I_w/I_z+((k*l*100)^2*G*I_t)/(\pi^2*E_s*I_z)+(C_2*z_g-C_3*z_j)^2)^{0,5}$	=	51,27 < 1
$P_3=$	$C_2*z_g-C_3*z_j$	=	12,65
$M_{cr}=$	$P_1*(P_2-P_3)/10^3$	=	471,09 kNm

Cross-section class 1 β_w= 1,00

$\lambda_{trans,LT}=$	$(\beta_w*W_{pl}*f_y/M_{cr}/10^3)^{0,5}$	=	1,595 < 1

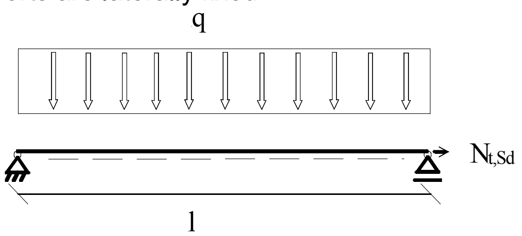
Apply strut curve c:

$\alpha_{LT}=$			0,49
$\varphi_{LT}=$	$0{,}5*(1+\alpha_{LT}*(\lambda_{trans,LT}-0{,}2)+\lambda_{trans,LT}^2)$	=	2,114
$\chi_{LT}=$	$1/(\varphi_{LT}+(\varphi_{LT}^2-\lambda_{trans,LT}^2)^{0,5})$	=	0,286
$M_{b,Rd}=$	$\beta_w*\chi_{LT}*W_{pl}*f_y/\gamma_M/10^3$	=	311,51 kNm

$M_{sd} / M_{b,Rd}$ = **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_{sd} =	18,00 kN/m
$N_{t,sd}$=	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=	$\sqrt{(235/f_y)}$	=	0,81

Partial safety factors:

γ_M=	1,10
γ_g=	1,35

A = $(b_o * t_o + b_u * t_u + h * s)$ = 515,00 cm²

Location of center of gravity:

e= $(b_u * t_u * (h + t_o) + h * s * (h + t_o) / 2) / A$ = 55,27 cm

Location of shear centre:

s_s= $(h + (t_o + t_u) / 2) * (b_u{}^3 / (b_o{}^3 + b_u{}^3))$ = 68,45 cm

Plastic neutral axis:

x = $(A / 2 - (b_o * t_o)) / s$ = 58,75 cm

Section classification As in Table 5.3.1:

Flange:

c = $((b_o - s_) / 2 - t_s * \sqrt{(2)}) / t_o$ = 5,02 cm

$c / (9 * \varepsilon)$ = 0,69 < 1

Section class 1

Web:

α = x / h = 0,59 cm

$h / s / (456 * \varepsilon / (13 * \alpha - 1))$ = 0,90 < 1

$h / s / (396 * \varepsilon / (13 * \alpha - 1))$ = 1,04 > 1

Section class 2

Check bending moment strength:

M_{sd} =	q_{sd} * l² / 8	=	1296,00 kNm
I_z =	t_o * b_o³ / 12 + t_u * b_u³ / 12	=	55125,00 cm4
I_y =	b_o*t_o*e²+b_u*t_u*(h+t_u-e)²+s*h³/12+h*s*(e-(h/2+t_u/2))²	=	1003886,210 cm^4
W_{com} =	I_y / (e + t_o / 2)	=	17605,86 cm³

Effective bending moment:

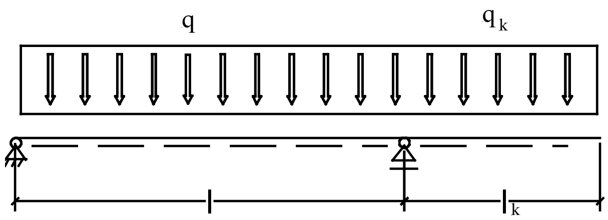
M_{sd} =	q_{sd} * l² / 8	=	1296,00 kNm
Assumption as in 5.5.3 ψ_{vec} =			0,80
$\sigma_{com,Ed}$ =	M_{sd} * 100 / W_{com} - ψ_{vec} * $N_{t,sd}$ / A	=	6,58 kN/cm²
$M_{eff,sd}$ =	W_{com} * $\sigma_{com,Ed}$ / 100	=	1158,47 kNm
As in TableF.1.2(2) k =			1,00
As in TableF.1.1(4) k_w =			1,00

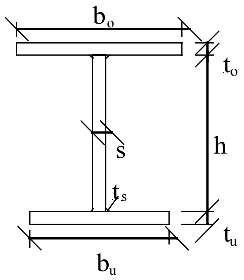
As in Table: F.1.2

C_1 =			1,132
C_2 =			0,459
C_3 =			0,525
z_s =	-(s_s - e)	=	-13,18 cm
I_t =	1/3 * (b_u * t_u³ + b_o * t_o³ + h * s³)	=	1552,92 cm^4
z_a =			0,00 cm
z_g =	z_a - z_s	=	13,18 cm
β_f =	b_o³ / (b_o³ + b_u³)	=	0,339 < 0,5
z_j =	(2 * β_f - 1) * (h + (t_o + t_u) / 2) / 2	=	-16,66 cm
I_w =	β_f*(1-β_f)*I_z*(h+(t_o+t_u)/2)²	=	0,1323*10^9 cm^6
P_1=	C_1*(π²*E_s*I_z/(k*l*10^3)²)/10	=	22,45 kNm
P_2=	(k/k_w)²*(I_w/I_z+((k*l*100)²*G*I_t)/(π²*E_s*I_z)+(C_2*z_g-C_3*z_j)²)^{0,5}	=	94,66
P_3=	C_2*z_g-C_3*z_j	=	14,80
M_{cr}=	P_1*(P_2-P_3)	=	1792,86 kNm
W_{pl} =	b_u*t_u*(x+t_u/2)+x²*s/2+b_o*t_o*(h-x+t_o/2)+(h-x)²*s/2	=	21760,63 cm³
Section class 2 β_w=			1,00
$\lambda_{trans,LT}$=	(β_w*W_{pl}*f_y/(M_{cr}*10³))^{0,5}	=	2,076
As in Table 5.5.1 α=			0,49
φ=	0,5*(1+α*($\lambda_{trans,LT}$-0,2)+$\lambda_{trans,LT}$²)	=	3,11
χ_{LT}=	1/(φ+(φ²-$\lambda_{trans,LT}$²)^{0,5})	=	0,1843
$M_{b,Rd}$=	β_w*χ_{LT}*W_{pl}*f_y/(γ_M*10)²/10	=	1176,63 kNm

$M_{eff,sd}$/$M_{b,Rd}$ = **<u>0,98 < 1</u>**

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
ε=	$\sqrt{(235/f_y)}$	=	1,00

Partial safety factors:

γ_{fg}=	1,35
γ_{fq}=	1,50
ψ=	0,90
γ_M=	1,10

Design loads:

q_{d1}=	$\gamma_{fg} * g + \gamma_{fq} * q$	=	91,65 kN/m
q_{d2}=	$\gamma_{fg} * g + \psi * \gamma_{fq} * (q + s_0)$	=	95,44 kN/m
q_d=	MAX(q_{d1}; q_{d2})	=	95,44 kN/m
M_{sdk}=	$-q_d * l_k^2 / 2$	=	-1193,00 kNm
B_v=	$(l + l_k)^2 / 2 * q_d / l$	=	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_{sdfeld}=	$A_v * (A_v / q_d / 2)$	=	786,41 kNm

Plastic neutral axis:

$A =$	$(b_o * t_o + b_u * t_u + h * s)$	=	250,00 cm²
$x =$	$(A / 2 - (b_o * t_o)) / s$	=	16,67 cm
$W_{pl} =$	$(b_o*t_o*(x+t_o/2)+0{,}5*s*x^2+b_u*t_u*(t_u/2+(h-x))+0{,}5*s*(h-x)^2)$	=	6043,33 cm³
$M_{pl} =$	$W_{pl} * f_y / 10^3$	=	1420,18 kNm

Section classification Kragarm As in Table 5.3.1:

Bottom Flange:

$c =$	$0{,}5 * (b_u - s) - t_s * \sqrt{(2)}$	=	13,12 cm
$c / t_u / (9 * \varepsilon)$		=	0,73 < 1

Web:

$\alpha =$	$(h - x) / h$	=	0,72
$((h - 2 * t_s * \sqrt{(2)}) / s)/(396 * \varepsilon / (13 * \alpha - 1))$		=	0,81 < 1

⇒ Section class 1.

Check shear strength:

$A_V =$	$h * s$	=	90,00 cm²
$V_{pl,Rd} =$	$(A_V * f_y) / (10 * \sqrt{(3)} * \gamma_M)$	=	1110,09 kN
$V_{sd} =$	$MAX(B_l; B_r ;A_V)$	=	614,68 kN
$V_{sd} / (V_{pl,Rd} * 0{,}5)$		=	**1,11 > 1**

⇒ Mpl, reduced.

Check bending moment strength:

$\rho =$	$(2 * V_{sd} / V_{pl,Rd} - 1)^2$	=	0,012
$M_{V,Rd} =$	$(1 - \rho) * M_{pl} / \gamma_M$	=	1275,58 kNm
$ABS(M_{sdk}) / M_{V,Rd}$		=	**0,94 < 1**

Section classification Feld As in Table 5.3.1:

topr Flange:

$c =$	$0{,}5 * (b_o - s) - t_s * \sqrt{(2)}$	=	23,12 cm
$c / t_o / (10 * \varepsilon)$		=	1,16 > 1
$c / t_o / (14 * \varepsilon)$		=	0,83 < 1

⇒ Section class 3.

Web:

$\alpha =$	x / h	=	0,28 < 1
$((h - 2 * t_s * \sqrt{(2)}) / s) / (396 * \varepsilon / (13 * \alpha - 1))$		=	0,26 < 1

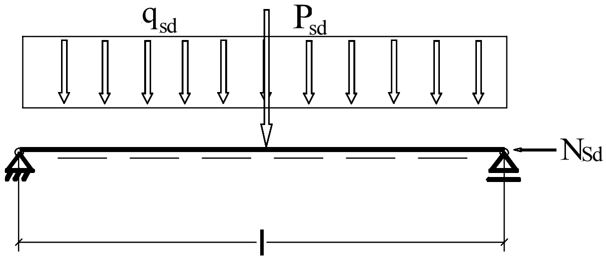
⇒ Section class 1.

Check section strength:

$ABS(M_{sdfeld}) / (M_{pl} / \gamma_M)$	=	**0,61 < 1**

Pos.: Analysis single-span beam:

End supports are laterally fixed.



Load diagram:

Span length l =	6,00 m

Loads:

q_{sd} =	10,00 kN/m
N_{sd}=	273,00 kN
P_{sd}=	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=	$\sqrt{(235/f_y)}$	=	1,00

Partial safety factors:

γ_M=	1,10
Plastic shape factor α_{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 cm^4
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} =	α_{pll} * W_{el}	=	634,98 cm³

Section classification As in Table 5.3.1:

Web:

$(h_1 / s) / (33 * \varepsilon)$ = 1,06 ~ 1

⇒ Section class 1.

Flange:

$(b / 2 / t) / (10 * \varepsilon)$ = 0,70 < 1

⇒ Section class 1.

Check bending moment strength:

M_{sd}=	q_{sd} * l² / 8 + P_{sd} * l / 4	=	67,50 kNm
$M_{pl,y,Rd}$=	W_{pl} * f_y / γ_M / 10³	=	135,65 kNm
$N_{pl,Rd}$=	A * f_y / γ_M / 10	=	1149,36 kN
n=	N_{sd} / $N_{pl,Rd}$	=	0,238
α=	(A - 2 * b * t / 10²) / A	=	0,403 < 0,5

Reduced limit moment due to tension force:

$M_{N,y,Rd}$=	$M_{pl,y,Rd}$ * (1 - n) / (1 - 0,5 * α)	=	129,45 kNm

M_{sd} / $M_{N,y,Rd}$ = **0,52 ≤ 1**

Check shear strength:

V_{sd}=	(P_{sd} + q_{sd} * l) / 2	=	37,50 kN
A_v=	1,04 * h * s / 10²	=	22,15 cm²
$V_{pl,Rd}$=	A_v * (f_y / √(3)) / γ_M / 10	=	273,20 kN

V_{sd} / $V_{pl,Rd}$ = **0,14 ≤ 1**

Where max M occurs::

V_s=	P_{sd} / 2	=	7,50 kN

V_s / (0,5 * $V_{pl,Rd}$) = **0,05 ≤ 1**

⇒ No interaction!

Check buckling strength:

λ_y=	100 * l / i_y	=	48,00 < 1
$\lambda_{trans,y}$=	λ_y / (π * √(E_s / f_y))	=	0,511
λ_z=	100 * l / i_z	=	179,10
$\lambda_{trans,z}$=	λ_z / (π * √(E_s / f_y))	=	1,91
h/b		=	2,00 > 1,2
t/10		=	1,07 < 4

Apply strut curve:

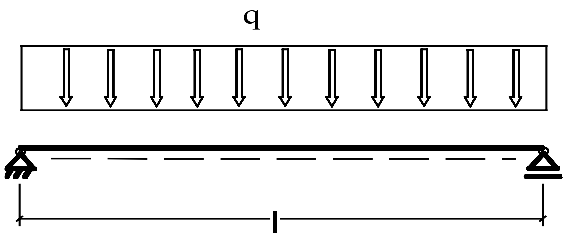
line =	SEL("steel/buck"; line;)	=	a
α =	TAB("steel/buck"; α; line=line)	=	0,210
χ_y =	TAB("steel/buck"; χ; α=α; λq=$\lambda_{trans,y}$)	=	0,9205

Auxilliary values:

β_M=	1,30 Bild 5,5,3		
μ_y=	$\lambda_{trans,y}$ * (2 * β_M - 4) + (W_{pl} - W_{el}) / W_{el}	=	-0,575 < 0,9
k_y=	1 - (μ_y * N_{sd}) / (χ_y * A * f_y)	=	1,013 < 1,5

N_{sd} / (χ_y * A * f_y / γ_M) + 100 * k_y * M_{sd} / (W_{pl} *f_y / γ_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
ε=	$\sqrt{(235/f_y)}$	=	1,00

Partial safety factors:

γ_{fg}=	1,35
γ_{fq}=	1,50
ψ=	0,90
γ_{Mo}=	1,10

Design loads:

q_{d1}=	γ_{fg}*g+γ_{fq}*q	=	11,55 kN/m
q_{d2}=	γ_{fg}*g+ψ*γ_{fq}*(q+s_0)	=	13,50 kN/m
q_d=	MAX(q_{d1};q_{d2})	=	13,50 kN/m

Design moment:

M_{sd}=	q_d*l²/8	=	94,92 kNm
Profil Typ =	SEL("steel/Profils"; Name;)	=	IPE
Selected Profil =	SEL("steel/"Typ; Name; Mplyd≥M_{sd})	=	IPE 270
with $M_{pl,yd}$ =	TAB("steel/"Typ; Mplyd; Name=Profil)	=	107,00 kNm

M_{sd} / $M_{pl,yd}$ = **0,89 < 1**

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:

$(h_1 / s) / (72 * \varepsilon)$ = **0,46 < 1**

Flange:

$(b / 2 / t) / (10 * \varepsilon)$ = **0,66 < 1**

⇒Section class 1.

Check shear strength:

V_{sd} = 0,5 * q_d * l = 50,63 kN

A_v = 1,04 * h * s / 100 = 18,53 cm²

$V_{pl,Rd}$ = $(A_v * f_y/10) / (\sqrt{(3)} * \gamma_{Mo})$ = 228,55 kN

$V_{sd} / V_{pl,Rd}$ = **0,22 ≤ 1**

Analysis working / servicing capacity as in Table 4.1:

q_{ser} = $g + \psi * (q + s_0)$ = 9,30 kN/m

δ_{max}= $5/384*(q_{ser}*(l*100)^4)/(E_s*I*10)$ = 3,15 cm

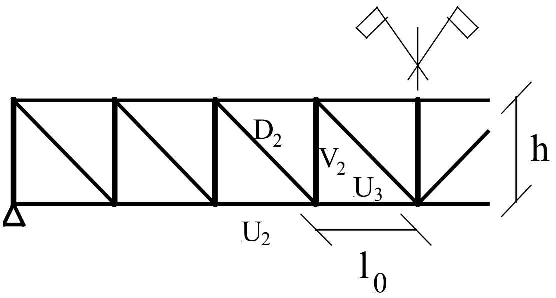
$\delta_{max}/(l*100/250)$ = **1,05 ≤ 1**

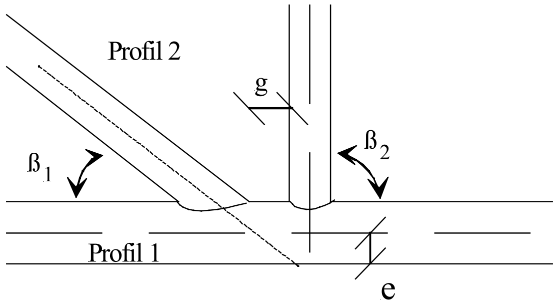
δ_2= $5/384*((q+s_0)*(l*100)^4)/(E_s*I*10)$ = 2,37 cm

$\delta_2/(l*100/300)$ = **0,95 ≤ 1**

Trusses and lattice girders

Lattice girder with circular hollow sectionsn:





Plan and elevation values:

Span length l_0 = 250,00 cm
β_1 = 30,96 °
β_2 = 90,00 °
h = 150,00 cm
g = 2,00 cm
e = 0,81 cm
Weld thickness a = 3,00 mm

Profil1:	SEL("steel/R"; Name;)	=	R 114.3x5
d_U =	TAB("steel/R"; d; Name=Profil1)	=	114,30 mm
t_U =	TAB("steel/R"; t; Name=Profil1)	=	5,00 mm
A_U =	TAB("steel/R"; A; Name=Profil1)	=	17,20 cm²
Profil2:	SEL("steel/R"; Name;)	=	R 60.3x2.9
d_2 =	TAB("steel/R"; d; Name=Profil2)	=	60,30 mm
t_2 =	TAB("steel/R"; t; Name=Profil2)	=	2,90 mm
A_2 =	TAB("steel/R"; A; Name=Profil2)	=	5,23 cm²
i_2 =	TAB("steel/R"; 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 = $\sqrt{(235 / f_y)}$ = 1,00
γ_M = 1,10
λ_1 = 93,90 * ε = 93,90

Member forces:

U_2 = 116,67 kN
U_3 = 200,00 kN
D_2 = 97,18 kN
V_2 = 50,00 kN Druck

$l_0 / (d_U / 10)$ = 21,87 > 6
$h / (d_2 / 10)$ = 24,88 > 6

Section classification:

Member V2:

$d_2 / t_2 / (50 * \varepsilon^2)$ = **0,42 < 1**

Section class 1

Analysis of the members:

Member V_2:

Section class 1 β_A = 1,00

λ = $0{,}75 * h/i_2$ = 55,42

λ_{trans} = $\lambda / \lambda_1 * \sqrt{(\beta_A)}$ = 0,590

Apply strut curve a

α = 0,21

φ = $0{,}5 * (1 + \alpha * (\lambda_{trans} - 0{,}2) + \lambda_{trans}^2)$ = 0,715

χ = $1 / (\varphi + (\varphi^2 - \lambda_{trans}^2)^{0,5})$ = 0,8937

$N_{b,Rd}$ = $\chi * \beta_A * A_2 * f_y / 10 / \gamma_M$ = 99,85 kN

$V_2 / N_{b,Rd}$ = **0,501 < 1**

Member D2:

N_{Rd} = $A_2 * f_y / 10 / \gamma_M$ = 111,73 kN

D_2 / N_{Rd} = **0,870 < 1**

Member U2;U3

N_{Rd} = $A_U * f_y / 10 / \gamma_M$ = 367,45 kN

U_3 / N_{Rd} = **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	=	20,793 > 10
d_2 / t_2	=	20,793 < 50
$(t_U + t_2) / g$	=	3,950 < 1

Analysis as given in Table. K.6:

$t_2 / 2{,}5$	=	1,160 > 1
$t_U / 25$	=	0,200 < 1
β = MIN(β_1; β_2)	=	30,960 °
$30 / \beta$	=	0,969 < 1
$e / (d_U / 4)$	=	0,028 < 1

⇒ Eccentricities can be neglected.

$a / (\mathrm{MIN}(t_U; t_2))$ = 1,034 > 0,84

Calculate maximum stress As in Table K.6

Collapse of top chord:

$\gamma = \quad d_U / (2 * t_U) \quad = \quad 11{,}43$

$10 * g / t_U \quad = \quad 4{,}00$

$c = \quad (0{,}5 * 10 * g / t_U - 1{,}33) \quad = \quad 0{,}670$

$k_g = \quad \gamma^{0{,}2} * (1 + 0{,}024 * \gamma^{1{,}2} / (1 + 2{,}7183^c)) \quad = \quad 1{,}874$

$k_p =$ 1,00 (Zug)

$\beta = \quad d_2 / d_U \quad = \quad 0{,}528$

$N_{1,Rd} = k_g * k_p * f_y * t_U^2/((1-\beta)*SIN(\beta_1))*(1{,}8+10{,}2*d_2/d_U)*(1{,}1/ 10^3 /\gamma_M) \quad = \quad 325{,}61$ kN

$N_{2,Rd} = SIN(\beta_1) / SIN(\beta_2) * N_{1,Rd} \quad = \quad 167{,}51$ kN

$V_2 / N_{2,Rd} \quad = \quad$ **0,30 < 1**

Collapse due to punching shear:

$d_2 / (d_U - 2 * t_U) \quad = \quad 0{,}58 < 1$

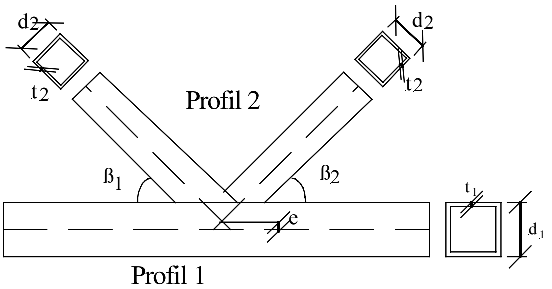
$N_{i1,Rd} = \quad f_y / \sqrt{(3)} * t_2 * \pi * d_2 * (1+SIN(\beta_1)) / (2*(SIN(\beta_1))^2) * (1{,}1 / 10^3 / \gamma_M) \quad = \quad 213{,}27$ kN

$D_2 / N_{i1,Rd} \quad = \quad$ **0,46 < 1**

$N_{i1,Rd} = \quad f_y / \sqrt{(3)} * t_2 * \pi * d_2 * (1+SIN(\beta_2)) / (2*(SIN(\beta_2))^2)*(1{,}1 / 10^3 / \gamma_M) \quad = \quad 74{,}54$ kN

$V_2 / N_{i1,Rd} \quad = \quad$ **0,67 < 1**

Truss joint with rectangular hollow sections:



Plan and elevation values:

Span length l_2 =	200,00 cm
Projection length λ_{ov}=	40,00 %
β_1 =	50,00 °
β_2 =	50,00 °
e =	-2,00 cm
Weld thickness a =	3,50 mm

Profil1:	SEL("steel/QR"; Name;)	=	QR 100x4.0
d_1 =	TAB("steel/QR"; a; Name=Profil1)	=	100,00 mm
t_1 =	TAB("steel/QR"; t; Name=Profil1)	=	4,00 mm
A_1 =	TAB("steel/QR"; A; Name=Profil1)	=	15,20 cm²
Profil2:	SEL("steel/QR"; Name;)	=	QR 60x4.0
d_2 =	TAB("steel/QR"; a; Name=Profil2)	=	60,00 mm
t_2 =	TAB("steel/QR"; t; Name=Profil2)	=	4,00 mm
A_2 =	TAB("steel/QR"; A; Name=Profil2)	=	8,82 cm²
i_2 =	TAB("steel/QR"; i; Name=Profil2)	=	2,28 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 =	$\sqrt{(235 / f_y)}$	=	1,00
γ_M =			1,10
λ_1 =	93,90 * ε	=	93,90

Member forces:

U_l =	250,00 kN
U_r =	150,00 kN
D_l =	77,80 kN (Druck)
D_r =	77,80 kN

Section classification:

Member Dl:

$d_2 / t_2 / (33 * \varepsilon^2)$ = **0,45 < 1**

Section class 1

Analysis of the members:
Member V2:

$\lambda =$	$0{,}75 * l_2 / i_2$	=	65,79
Section class 1 $\beta_A =$			1,00
$\lambda_{trans} =$	$\lambda / \lambda_1 * \sqrt{(\beta_A)}$	=	0,70
Apply strut curve a			
$\alpha =$			0,21
$\varphi =$	$0{,}5 * (1 + \alpha * (\lambda_{trans} - 0{,}2) + \lambda_{trans}{}^2)$	=	0,797
$\chi =$	$1 / (\varphi + (\varphi^2 - \lambda_{trans}{}^2)^{0,5})$	=	0,8489
$N_{b,Rd} =$	$\chi * \beta_A * A_2 * f_y / 10 / \gamma_M$	=	159,956 kN

$D_l / N_{b,Rd}$ = **<u>0,486 < 1</u>**

Member Dr:

$N_{Rd} =$	$A_2 * f_y / 10 / \gamma_M$	=	188,427 kN

D_r / N_{Rd} = **<u>0,413 < 1</u>**

Member U2;U3

$N_{Rd} =$	$A_1 * f_y / 10 / \gamma_M$	=	324,727 kN

U_l / N_{Rd} = **<u>0,770 < 1</u>**

Analysis of joint:

Check permissible stress as in Table: K.14

d_2 / d_1	=	0,60 > 0,25
$d_2 / t_2 / (1{,}1 * \sqrt{(E_s / f_y)})$	=	0,456 < 1 (Druck)
d_2 / t_2	=	15,000 < 35 (Zug)
d_1 / t_1	=	25,000 < 35 (Zug)
$\lambda_{ov} / 100$	=	0,400 < 1
$25 / \lambda_{ov}$	=	0,625 < 1

Analysis as given in Table. K.12:

$t_2 / 2{,}5$		=	1,600 > 1
$t_2 / 25$		=	0,160 < 1
$\beta =$	$MIN(\beta_1; \beta_2)$	=	50,000 °
$30 / \beta$		=	0,600 < 1
$e / (-d_1 * 0{,}55)$		=	0,036 < 1

$\Rightarrow$ Eccentricities can be neglected.

$a / MIN(t_1; t_2)$	=	0,875 > 0,84

Calculate maximum stress As in Table K.14

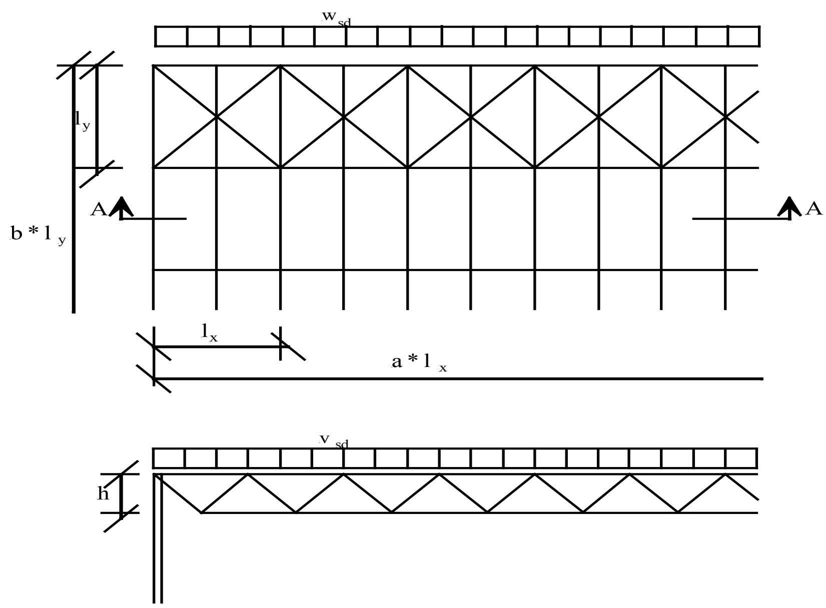
Only the overlapping diagonals will be analysed:

$b_{eff} =$	$1 / (d_1 / t_1) * t_2 / t_1 * d_2$	=	2,40 cm
$b_{eff} / (MIN(d_1; d_2)/10)$		=	0,40 < 1
$b_{e,ov} =$	$1 / (d_2 / t_2) * t_1 / t_2 * d_2$	=	4,00 cm
$b_{e,ov} / (MIN(d_1; d_2)/10)$		=	0,67 < 1
$k_n =$			1,00 (Zug)
so that			
$\lambda_{ov} / 50$		=	0,80 < 1
$25 / \lambda_{ov}$		=	0,63 < 1

$N_{2,Rd} = f_y/10^2 * t_2 * (\lambda_{ov} / 50 * (2 * d_2 - 4 * t_2)/10 + b_{eff} + b_{e,ov})$ = 138,368 kN

$D_r / N_{2,Rd}$ = **<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_{sd} =	2,00 kN/m
v_{sd} =	5,00 kN/m

Design loads:

M_{sd} =	v_{sd} * (a * l_x)² / 8	=	562,50 kNm
Compression force in the top chord:			
N =	M_{sd} / h	=	281,25 kN
Total compression force / Stabiilising force:			
N_{ges} =	(a + 1) / 2 * N	=	1546,88 kN
Imperfection:			
k_{r1} =	√(0,2 + 1 / ((a + 1) / 2))	=	0,62
k_{r2} =			1,00
k_r =	MIN(k_{r1}; k_{r2})	=	0,62 < 1
e_0 =	k_r * a * l_x / 5	=	3,72 cm
Equivalent load instead of buckling:			
δ_q =	a * l_x / 25	=	1,20 cm
q =	N_{ges} / (60 * a * l_x) * (k_r + 0,2)	=	0,70 kN/m
h_{sd} =	w_{sd} + q	=	2,70 kN/m

Check deformation δq:

$E'_{Ieff} =$ $10^3 * E_s * 0{,}5 * A * l_y^2$ = $8{,}24*10^{10}$ kNcm²

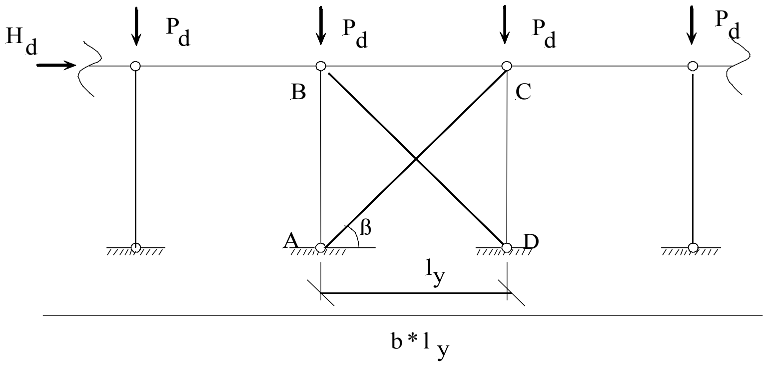
Deformation in the middle of truss:

$\delta_{q1} =$ $(5/384 * h_{sd} *((a * l_x)^4) / (E'I_{eff}/10^6))$ = 0,35 cm

δ_{q1} / δ_q = **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 β = ATAN(h/l_y) = 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 cm^4
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 cm^4
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/R"; Name;)	=	R 177.8x5
d =	TAB("steel/R"; d; Name=ProfilR)	=	177,80 mm
t_r =	TAB("steel/R"; t; Name=ProfilR)	=	5,00 mm
A_r =	TAB("steel/R"; A; Name=ProfilR)	=	27,10 cm²
i_r =	TAB("steel/R"; i; Name=ProfilR)	=	6,11 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
ε =	$\sqrt{(235 / f_y)}$	=	1,00
γ_M =			1,10
λ_1 =	93,90 * ε	=	93,90

Check swaying:

k =	I_{y1} / I_{y2}* h / ly	=	0,644
Deflection at top of frame due to horizontal load:			
H =			1,00 kN
δ_F =	(100 * h)³ * 10 / (12 * E_s * I_{y2}) * (2 * k + 1) / k * H	=	0,702 cm
Shear strength:			
S_v =	2 / 10 * n * E_s * A_r * h * l_y^2 / l_r^3	=	430,1*10^3 kN
δ_w =	H * h / S_v * 100	=	1,16*10^{-3} cm
Deflection at top of frame:			
H =			1,00 kN
δ_{tot} =	1 / (1 / δ_F + 1 / δ_w)	=	1,158*10^{-3} cm
Reduction due to bracing:			
(1 - (δ_{tot} / δ_F)) * 100		=	99,84 % > 80

⇒ Non sway frame.

Analysis of vertical bracing

Imperfection to 5.2.4.3:

φ_0 =	1 / 200	=	0,005
k_{c1} =	$\sqrt{(0,5 + 1 / n_c)}$	=	0,82
k_{c2} =			1,00
k_c =	MIN(k_{c1}; k_{c2})	=	0,82
k_{s1} =	$\sqrt{(0,2 + 1 / n_s)}$	=	1,10
k_{s2} =			1,00
k_s =	MIN(k_{s1}; k_{s2})	=	1,00
φ =	φ_0 * k_c * k_s	=	0,0041
ΔH_d =	φ * P_d * (b + 1)	=	7,38 kN
H_{sd} =	H_d + ΔH_d	=	87,38 kN
Bracing is restrained out of plane of frame			
δ_w / (h * 100) * (b + 1) * P_d / H		=	0,004 < 0,1

⇒ restrained from moving out of plane.

Analysis of lacing angles:

Design compression load:

N_{sd} =	$H_{sd} / (2 * \cos(\beta))$	=	56,88 kN

Slenderness:

λ =	$100 * l_r / i_r$	=	127,82

Effective slenderness:

λ_{trans} =	$\lambda / (\lambda_1 * \varepsilon)$	=	1,36

Apply strut curve a:

α =			0,21
β_A =			1,000
φ =	$0,5 * (1 + \alpha * (\lambda_{trans} - 0,2) + \lambda_{trans}^2)$	=	1,547
χ =	$1 / (\varphi + (\varphi^2 - \lambda_{trans}^2)^{0,5})$	=	0,4378
N_{Rd} =	$\chi * \beta_A * A_r * f_y / 10 / \gamma_M$	=	253,47 kN
N_{sd} / N_{Rd}	=	**0,224 < 1**	

Analysis of frame:

Column CD:

N_{sd} =	$P_d + H_{sd} / 2 * h / l_y$	=	336,41 kN

Buckling in the plane of frame:

k_c =	$I_{y2} / h / 100$	=	5,02 cm³
k_1 =	$1,0 * I_{y1} / l_y / 100$	=	3,23 cm³
k_2 =			0,0
η_2 =			1,0
η_1 =	$k_c / (k_c + k_1 + k_2)$	=	0,61

Slenderness ratio as in Fig. E.2.1.:

l =	h * 0,85	=	4,25 m
λ_y =	$100 * l / i_y$	=	57,05
λ_{trans} =	$\lambda_y / (\lambda_1 * \varepsilon)$	=	0,61
h_t / b_t		=	0,95 < 1,2

Apply strut curve b:

α =			0,34
β_A =			1,000
φ =	$0,5 * (1 + \alpha * (\lambda_{trans} - 0,2) + \lambda_{trans}^2)$	=	0,756
χ_y =	$1 / (\varphi + (\varphi^2 - \lambda_{trans}^2)^{0,5})$	=	0,8315

Buckling outside the plane of frame

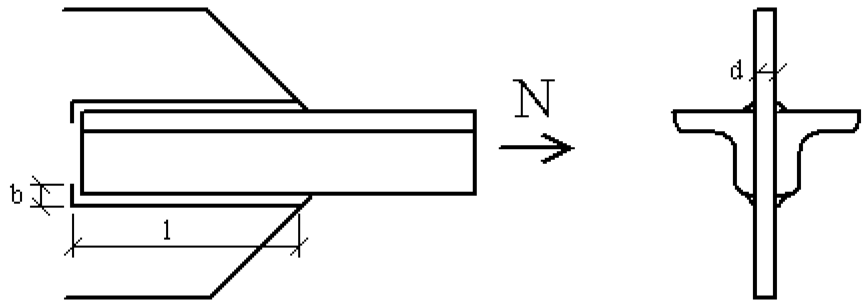
λ_y =	$100 * h / i_z$	=	110,62
λ_{trans} =	$\lambda_y / (\lambda_1 * \varepsilon)$	=	1,18

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 =	$1 / (\varphi + (\varphi^2 - \lambda_{trans}^2)^{0,5})$	=	0,4436
χ_{min} =	$MIN(\chi_y; \chi_z)$	=	0,4436
N_{Rd} =	$\chi_{min} * \beta_A * A_t * f_y / 10 / \gamma_M$	=	429,30 kN
N_{sd} / N_{Rd}	=	**0,784 < 1**	

Welded connections

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} = $2 * A * f_y / 10 / \gamma_{M0}$ = 295,25 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

β_{wl1} =	$1,2 - 0,2 * l / (150 * a_w)$	=	1,17
β_w =	$MIN(1; \beta_{wl1})$	=	1,00
l =	$\beta_w * l$	=	100,00 mm
$N_{w,Rd}$ =	$4 * l * a_w * f_u / (\sqrt{(3)} * \beta_w * \gamma_{Mw})/10^3$	=	266,04 kN

Maximum strength of gusset:

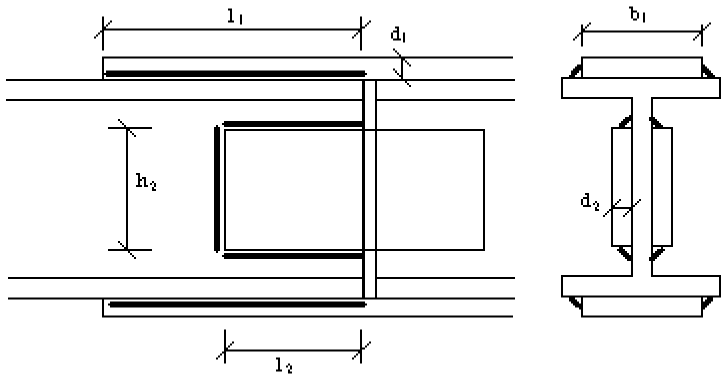
Assumption: Stress spreads at 30° angle in gusset:

A =	(2 * l * TAN(30) + h) * d / 100	=	17,55 cm²
N_{Rd} =	$A * f_y / 10 / \gamma_{M0}$	=	374,93 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_{sd} =	157,95 kNm
V_{sd} =	60,75 kN

Check beam:

$M_{c,Rd}$ =	$W_{pl} * f_y / \gamma_{M0} / 10^3$	=	282,51 kNm
$M_{sd} / M_{c,Rd}$		=	**<u>0,56 < 1</u>**
A_v =	1,04 * h * s / 100	=	35,78 cm²
$V_{pl,Rd}$ =	$A_v * f_y / 10 / \sqrt{(3)} / \gamma_{M0}$	=	441,32 kN
$V_{sd} / V_{pl,Rd}$		=	**<u>0,14 < 1</u>**

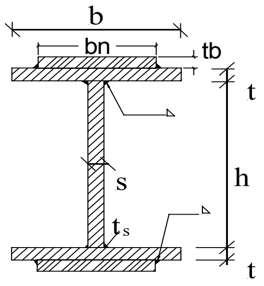
Check welds in flanges:

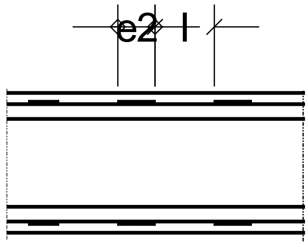
I_w =	$2 / 12 * d_2 * h_2^3 / 10^4$	=	2926,93 cm^4
I_f =	$2 * b_1 * d_1 * ((h + d_1) / 2)^2 / 10^4$	=	28224,00 cm^4
M_w =	$M_{sd} * I_w / (I_w + I_f)$	=	14,84 kNm
M_f =	$M_{sd} * I_f / (I_w + I_f)$	=	143,11 kNm
Effective tension force in flange			
N_{sd} =	$M_f / (h + d_1) * 10^3$	=	340,74 kN
Limit shear stress of flange plates:			
$f_{v,wd}$ =	$f_u / 10 / (\sqrt{(3)} * \beta_w * \gamma_{Mw})$	=	20,78 kN/cm²
Design stress:			
f_{sd} =	$N_{sd} / (l_{wf} * a_{wf}) * 10^2$	=	11,83 kN/cm²
$f_{sd} / f_{v,wd}$	= **<u>0,57 < 1</u>**		

Change welds in web:

Centre of gravity			
e =	$(2 * l_2 * l_2 / 2 * a_{ws}) / (l_{ws} * a_{ws})$	=	58,82 mm
I_x =	$(a_{ws} * h_2^3 / 12 + 2 * l_2 * (h_2 / 2)^2 * a_{ws}) / 10^4$	=	3867,73 cm^4
I_y =	$(h_2 * a_{ws} * e^2 + 2 / 12 * a_{ws} * l_2^3 + 2 * l_2 * a_{ws} * (l_2 / 2 - e)^2) / 10^4$	=	1192,16 cm^4
Design moment			
M_{sw} =	$M_w + V_{sd} * (l_2 - e) / 10^3$	=	23,42 kNm
Maximum shear stress			
f_{sx} =	$10 * M_{sw} * h_2 / (4 * (I_x + I_y))$	=	3,24 kN/cm²
f_{sy} =	$(V_{sd} / (l_{ws} * a_{ws} / 100) + (100 * M_{sw} * ((l_2 - e) / 10)) / (2 * (I_x + I_y)))$	=	5,50 kN/cm²
f_s =	$\sqrt{(f_{sx}^2 + f_{sy}^2)}$	=	6,38 kN/cm²
$f_s / f_{v,wd}$	= **<u>0,31 < 1</u>**		

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_{Sd} =	241,90 kNm
V_{Sd} =	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=	$\sqrt{(235/f_y)}$	=	1,00

Partial safety factors:

γ_M =	1,10
γ_{Mw} =	1,25

Design:

W_{pl} =	2 * (t * b * (h+t)/2 + s * h²/8)	=	907,04 cm³
M_{pl} =	0,001 * W_{pl} * f_y / γ_M	=	193,78 kNm
$V_{pl,Rd}$ =	h * s * f_y/ 10 /(√(3) * γ_M)	=	335,49 kN

Distance between cover plates:

b_n =	(γ_M*10*M_{Sd}/f_y-W_{pl})/(2*t_b*(h/2+t+t_b/2))	=	-30,10 cm
sel b_n =			7,60 cm
W_{pl} =	2*(t*b*(h+t)/2+s*h²/8) +2*t_b*(h/2+t+t_b/2)*b_n	=	1133,22 cm³
$M_{pl,Rd}$ =	0,001 * W_{pl} * f_y / γ_M	=	242,10 kNm

Analysis:

M_{Sd} / $M_{pl,Rd}$	=	**1,00 ≤ 1**
V_{Sd} / (1/2 * $V_{pl,Rd}$)	=	**0,58 ≤ 1**

Length e2 of weld:

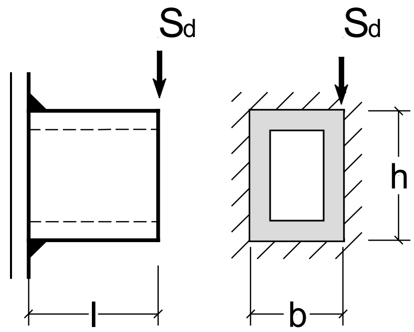
I_y =	(b*(h+2*t)³-(b-s)*h³)/12+2*t_b*b_n*(h/2+t+t_b/2)²	=	18726,53 cm^4
T' =	V_{Sd} * (t_b * b_n * (h/2+t+t_b/2)) / I_y	=	0,59 kN/cm
$f_{v,wd}$ =	f_u / 10 / (√(3)*β_w*γ_{Mw})	=	20,78 N/mm²

$F_{w,Rd}$ =	$0,2 * a * f_{v,wd}$	=	1,25 kN/cm
e_2 =	$T' / F_{w,Rd}$	=	0,47 cm
sel e2 =	**40,00 mm (Minimum allowed length)**		
T =	$T' * (e_2 + l)$	=	29,20 kN
$F_{w,Rd}$ =	$0,2*a*e_2*f_{v,wd}$	=	49,87 kN

Analysis:

$T / F_{w,Rd}$ = **<u>0,59 ≤ 1</u>**

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 cm²
I_y =	$(2 * b * a_w * (h/2)^2 + 2 * a_w * h^3 / 12)/ 10^4$	=	744,80 cm⁴
I_z =	$(2 * h * a_w * (b/2)^2 + 2 * a_w * b^3 / 12)/ 10^4$	=	320,00 cm⁴
I_p =	$I_y + I_z$	=	1064,80 cm⁴

Design load in relation to the centroid of weld:

S_{dz} =	S_d	=	100,00 kN
$M_{y,Sd}$ =	$S_d * l/10$	=	2000,00 kNcm
$M_{t,Sd}$ =	$S_d * b/20$	=	400,00 kNcm

Check stress at weakest point:

σ_A =	$M_{y,Sd} / I_y * h / 20$	=	18,80 kN/cm²
τ_1 =	$M_{t,Sd} / I_p * h / 20$	=	2,63 kN/cm²
τ_2 =	$M_{t,Sd} / I_p * b / 20 + S_{dz} / A$	=	5,29 kN/cm²
$\Rightarrow \sigma_{res}$ =	$\sqrt{(\sigma_A^2+\tau_1^2+\tau_2^2)}$	=	**19,71 kN/cm²**

Check weld strength:

Limit shear stress

$f_{w,Rd}$ =	$f_u / 10 / (\sqrt{(3)} * \beta_w * \gamma_{Mw})$	=	20,78 kN/cm²
$\sigma_{res} / f_{w,Rd}$		=	**0,95 < 1**