

SCW constructions
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Pasūtītājs : **SIA "LIEPĀJAS REĢIONĀLĀ SLIMNĪCA", Reģ. Rr. 42103041306**

Būvobjekta nosaukums : **Telpu pielāgošana magnētiskās rezonanses iekārtas
uzstādīšanai, Slimnīcas ielā 25, Liepājā**

Būves adrese : **Slimnīcas 25, Liepāja, LV-3414**

Dokumentācija : **Aprēķina atskaite**

Stadija : PR

Datums : 2026. gada 10. maijs

Būvprojekta daļas vadītājs: **Ilgvars Štāls**

Būvpr.sertif.Nr. **3-02054**

Aizpute 2026

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Izejas dati projektēšanai

Darba uzdevums:

Pagrabstāva pārseguma pastiprinājums magnētiskās rezonanses iekārtas uzstādīšanai neietekmējot būves nesošās konstrukcijas. Aprēķina atskaides noformēšana.

Vispārīgi projektēšanas nosacījumi

Apkurināta ēka, konstrukcijas atrodas iekštelpās.

Konstrukciju gabarītus skatīt PR BK rasējumos, pamatkomplekta rasējumu lapu uzskaitījums Vispārīgo rādītāju un Satura lapā (datēts ar 07.05.2026.)

Izmantotie būvnormatīvi un standarti

Projektēšana veikta un tai atbilstošie aprēķini veikti pamatojoties uz Eirokodeksu prasībām:

- LR Būvniecības likums;
- MK 2014.gada 19. augusta noteikumi nr.500 "Vispārīgie būvnoteikumi";
- LVS EN 1990:2006 "Eirokodekss. Konstrukciju projektēšanas pamatprincipi";
- LVS EN 1991-... "1. Eirokodekss. Iedarbes uz konstrukcijām.";
- LVS EN 1992-... "2. Eirokodekss. Betona konstrukciju projektēšana.";
- LVS EN 1993-... "3. Eirokodekss. Tērauda konstrukciju projektēšana.";
- LVS EN 1997-... "7. Eirokodekss. Ģeotehniskā projektēšana.";
- LBN 003-19 - "Būvklimatoloģija";
- LBN 207-15 - "Ģeotehniskā projektēšana";
- LBN 203-15 - "Betona būvkonstrukciju projektēšana";
- LBN 204-14 - "Tērauda būvkonstrukciju projektēšana";
- LBN 201-15 - "Būvju ugunsdrošība"

Statiskie aprēķini

Statiskie aprēķini veikti izmantojot datorprogrammu **AxisVM X7**. Aprēķina programmā AxisVM X7 izveidots pastiprinājuma konstrukcijas 3D modelis.

Kolonām un sijām ir stingrie savienojumi. Uz balstkonstrukcijas darbojas tikai vertikālā izkliedētā slodze. Slodzes izkliedē nodrošina dobre pārseguma paneli.

Izejot no būves projekta datiem ēkas esošās pamatnes nestspēja ir pietiekama.

Pieņemtie konstrukciju materiāli:

Pamati:

Betons - stiprības klase C30/37.

Stiegrojums - B500B.

Vides iedarbības klase – XC2

Tērauda rāmis:

Tērauds – S355

Izmantotās skrūves, uzgriežņi, paplāksnes – 8.8 klase

Iekārtas slodzes:

- Konstrukciju un apdares slāņu pašsvars;
- Lietderīgās slodzes;

Drošuma koeficienti:

- Pastāvīgās slodzes – 1.35;
- Mainīgās slodzes – 1.5.

Aprēķinu gaita:

- Izejas datu savākšana;
- Aprēķinu modeļa izveide un aprēķins programmā AXIS VM X7;
- Piepūļu un pārvietojumu analīze, elementu šķērs griezumu pārbaude;
- Materiālu apjomu kopsavilkums;
- Aprēķinu atskaites noformēšana.

Pasūtītāja definētā slodžu tabula

Ja grīdas virsmas spiediena izturība ir $\geq 150 \text{ N/cm}^2$ [1.5 N/mm^2], 3. paliktņi nav obligāti, tomēr Philips Healthcare iesaka izmantot grīdas paliktņus. Ar paliktņiem vai bez tiem, grīdas segums joprojām ir jānogriež ap vietu, kur atradīsies magnēta kājiņas, lai novērstu magnēta iegrimšanu grīdas segumā.

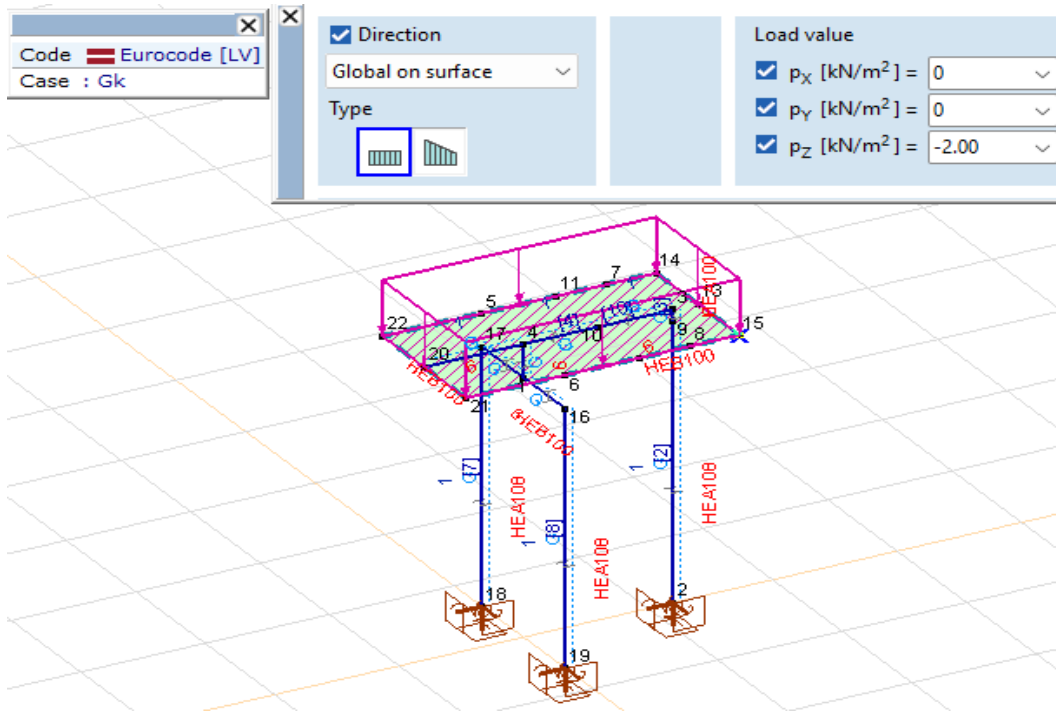
Grīdas slodze	1. un 2. pēda (katrai pēdai)	3. bloks (katrai pēdai)
Horizontālā grīdas slodze	4.0 kN	1.5 kN
Uz augšu vērsta slodze	10kN katrai skrūvei, vai 16.0 kN pēdai	N//A
Uz leju vērsta slodze	2.5 kN pēdai (5 kN kopā abām pēdām, ieskaitot pacienta svaru)	11 kN uz pēdu (37 kN uz 4 pēdām)

Metāla rāmja konstruktīvā slodze ievērtēta programmā

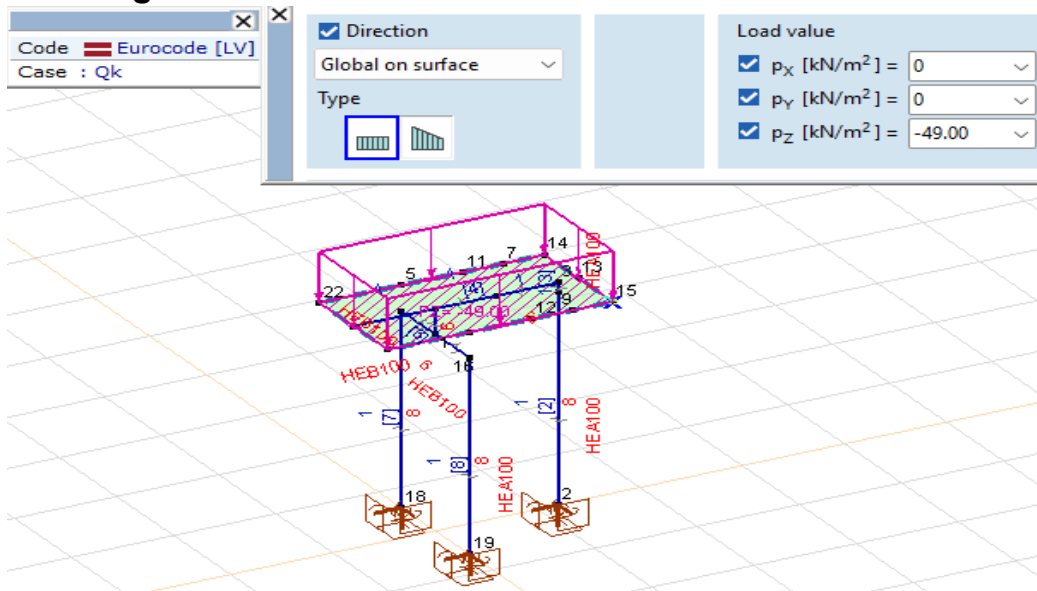
Pielikumi.

Pastiprinājuma konstrukcija

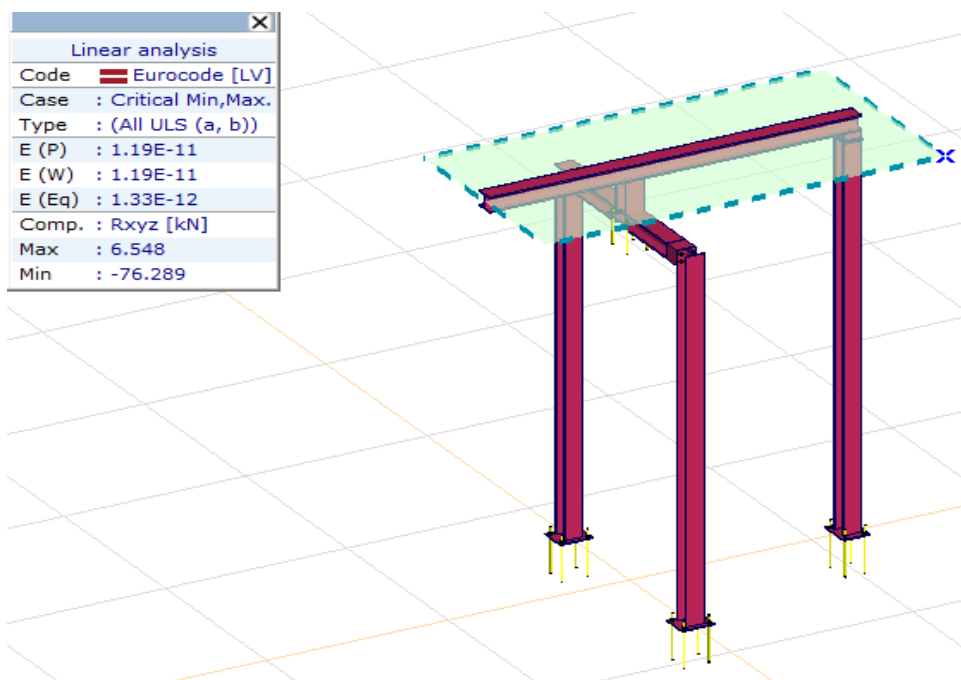
Konstruktīvā slodze



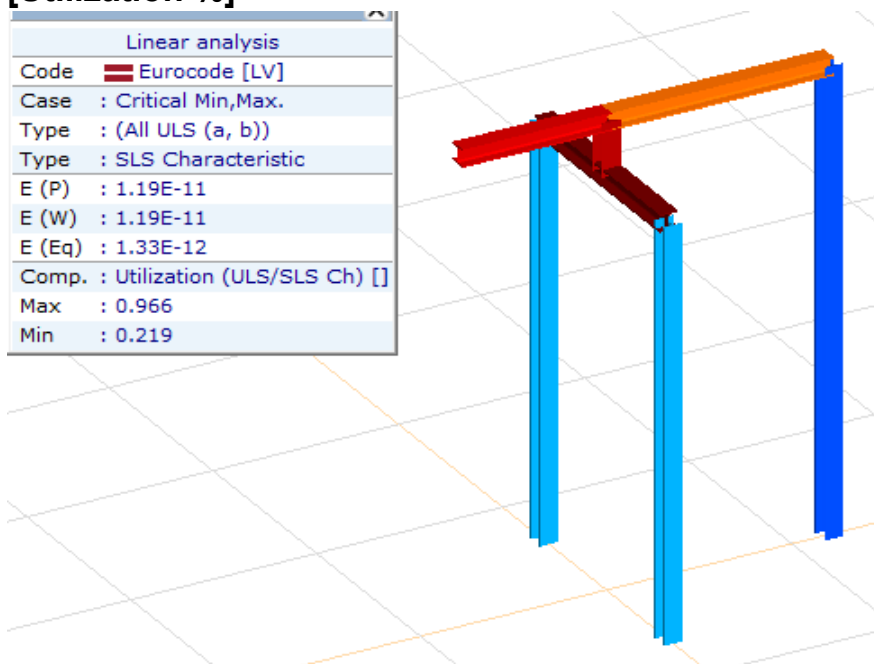
Lietderīgā slodze



Aprēķinu modeļa vizualizācija



Pastiprinājuma konstrukcijas izmantojamības pakāpe (ULS/SLS) [Utilization %]



Pastiprinājuma konstrukcija atbilst projekta nestspējas kritērijiem!



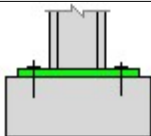
Magnētiskās rezonanses iekārtas uzstādīšana Slimnīcas ielā 25, Li

Analysis by SCW constructions

 Model: **Pastiprinājums.axs**

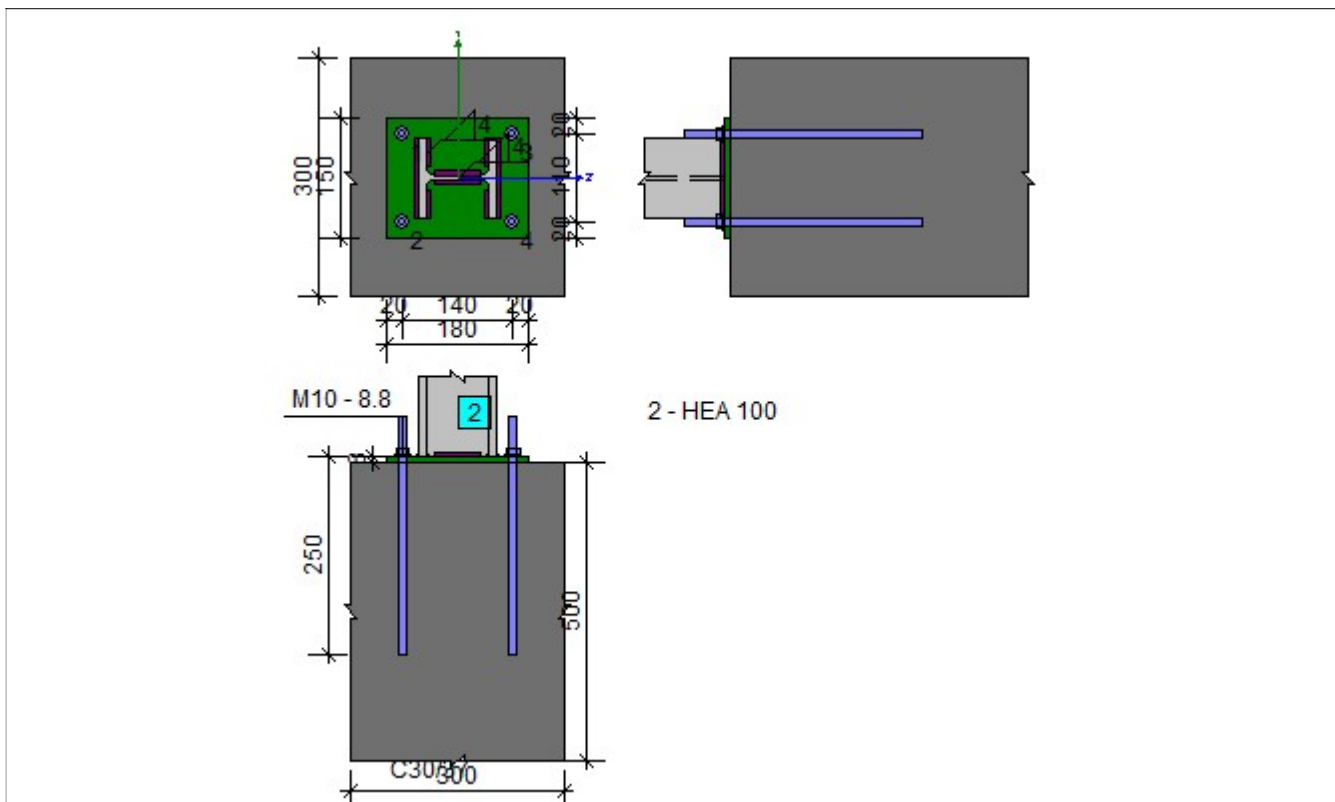
2026-05-13

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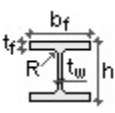
	Column base EC3 1991-1-8: 2008, EN 1992-4:2018	Ratio : 0.203	
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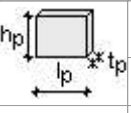
Structure node: 2

Structure members: 2



Data

Column HEA 100					
	h_c	b_{fc}	t_{fc}	t_{wc}	R_c
	96.00mm	100.00mm	8.00mm	5.00mm	12.00mm
	A_c	J_{y0c}	J_{z0c}	y_{0c}	z_{0c}
	21.24cm ²	349.24cm ⁴	133.81cm ⁴	50.00mm	48.00mm
Material	Grade	f_y	f_u		
	S 355	355.00MPa	490.00MPa		

Base plate			
	l_p	h_p	t_p
	180.00mm	150.00mm	8.00mm
Material	Grade	f_y	f_u
	S 355	355.00MPa	490.00MPa



Magnētiskās rezonanses iekārtas uzstādīšana Slimnīcas ielā 25, Lie

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Anchorage

Grade		8.8
Yield point	$f_y =$	640.00MPa
Resistance on tension	$f_u =$	800.00MPa
Bolt diameter	$d =$	10.00mm
Bolt opening diameter	$d_0 =$	11.00mm
Area of bolt section	$A =$	0.79cm ²
Effective section area of a bolt	$A_s =$	0.58cm ²
Number of rows	$n_y =$	2.00
Number of rows	$n_z =$	2.00
Vertical spacing of rows		$p_{y1} = 110.00mm$
Vertical spacing of rows		$p_{z1} = 140.00mm$

Welds

Thickness of fillet welds connecting beam flanges and front plate	$a_{fc} =$	4.00mm
Thickness of fillet welds connecting beam web and front plate	$a_{wc} =$	4.00mm

Foundation			
	L_{1f}	B_{1f}	H_{1f}
	300.00mm	300.00mm	500.00mm
Material	Grade	E_c	f_{ck}
	C30/37	32800.00MPa	30.00MPa

Materials coeffs

Coefficient	$\gamma_{M0} =$	1.000
Coefficient	$\gamma_{M2} =$	1.250
Coefficient	$\gamma_C =$	1.500
Coefficient	$\gamma_{Inst} =$	1.200

Forces

Design loads

Design loads [1.35*0.85*Gk] {1.5*Qk}



Magnētiskās rezonanses iekārtas uzstādīšana Slimnīcas ielā 25, Lie

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Axial force	$N_{Ed} =$	$-49.69kN$	
Shear force	$V_{y,Ed} =$	$0.13kN$	
Shear force	$V_{z,Ed} =$	$0.00kN$	
Bending moment	$M_{y,Ed} =$	$0.00kNm$	
Bending moment	$M_{z,Ed} =$	$-0.26kNm$	

In this version only the most external anchor row on tension side is taking into account for bending moment resistance of joint

Results

Resistance of a column base in the tension zone

Tensile resistance of a bolt

$$F_{t,Rd} = \frac{0.85 \cdot k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.85 \cdot 0.900 \cdot 800.00MPa \cdot 0.58cm^2}{1.250} = 28.40kN$$

Pull-out failure

Design tensile resistance

$$f_{ctd} = \frac{0.7 \cdot 0.3 \cdot f_{ck}^{\frac{2}{3}}}{\gamma_C} = \frac{0.7 \cdot 0.3 \cdot (30.00MPa)^{\frac{2}{3}}}{1.500} = 1.35MPa$$

Coefficient related to the quality of the bond conditions and concreting conditions

$$\eta_1 = 1.000$$

Coefficient related to the member diameter

$$\eta_2 = 1.000$$

Design value of the ultimate bond stress

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 2.25 \cdot 1.000 \cdot 1.000 \cdot 1.35MPa = 3.04MPa$$

Design anchorage length

$$L_{eff} = 250.00mm$$

Design uplift capacity

$$F_{btd} = \pi \cdot d \cdot L_{eff} \cdot f_{bd} = \pi \cdot 10.00mm \cdot 250.00mm \cdot 3.04MPa = 23.89kN$$

Geometrical parameters

Bolt distance from outer edge

$$e_{ep} = 20.00mm$$

Bolt distance from beam flange



$$m_x = 0.5 \cdot (p_{1z} - h_c) - 0.8 \cdot \sqrt{2} \cdot a_{fc} = 0.5 \cdot (140.00\text{mm} - 96.00\text{mm}) - 0.8 \cdot \sqrt{2} \cdot 4.00\text{mm} = 17.47\text{mm}$$

Bolt distance from outer horizontal edge plate

$$e_x = e_1 = 20.00\text{mm}$$

Effective length for one bolt in circular patterns

$$l_{eff,cp,1} = 2 \cdot \pi \cdot m_x = 2 \cdot \pi \cdot 17.47\text{mm} = 109.80\text{mm}$$

$$l_{eff,cp,2} = \pi \cdot m_x + 2 \cdot e = \pi \cdot 17.47\text{mm} + 2 \cdot 20.00\text{mm} = 94.90\text{mm}$$

$$l_{eff,cp,3} = \pi \cdot m_x + w = \pi \cdot 17.47\text{mm} + 110.00\text{mm} = 164.90\text{mm}$$

$$l_{eff,cp} = \min(l_{eff,cp,1}; l_{eff,cp,2}; l_{eff,cp,3}) = \min(109.80\text{mm}; 94.90\text{mm}; 164.90\text{mm}) = 94.90\text{mm}$$

Effective length for one bolt in non-circular patterns

$$l_{eff,nc,1} = 4 \cdot m_x + 1.25 \cdot e_x = 4 \cdot 17.47\text{mm} + 1.25 \cdot 20.00\text{mm} = 94.90\text{mm}$$

$$l_{eff,nc,2} = e + 2 \cdot m_x + 0.625 \cdot e_x = 20.00\text{mm} + 2 \cdot 17.47\text{mm} + 0.625 \cdot 20.00\text{mm} = 67.45\text{mm}$$

$$l_{eff,nc,3} = 0.5 \cdot b_p = 0.5 \cdot 150.00\text{mm} = 75.00\text{mm}$$

$$l_{eff,nc,4} = 0.5 \cdot w + 2 \cdot m_x + 0.625 \cdot e_x = 0.5 \cdot 110.00\text{mm} + 2 \cdot 17.47\text{mm} + 0.625 \cdot 20.00\text{mm} = 102.45\text{mm}$$

$$l_{eff,nc} = \min(l_{eff,nc,1}; l_{eff,nc,2}; l_{eff,nc,3}; l_{eff,nc,4}) = \min(94.90\text{mm}; 67.45\text{mm}; 75.00\text{mm}; 102.45\text{mm}) = 67.45\text{mm}$$

Effective length for one bolt for 1 mode

$$l_{eff,1} = \min(l_{eff,cp}; l_{eff,nc}) = \min(94.90\text{mm}; 67.45\text{mm}) = 67.45\text{mm}$$

Effective length for one bolt for 2 mode

$$l_{eff,2} = l_{eff,nc} = 67.45\text{mm}$$

Model 1: Complete yielding of the end plate

$$M_{pl,1,Rd} = \frac{0.25 \cdot l_{eff,1} \cdot t_p^2 \cdot f_{yp}}{Y_{M0}} = \frac{0.25 \cdot 67.45\text{mm} \cdot (8.00\text{mm})^2 \cdot 355.00\text{MPa}}{1.000} = 0.38\text{kNm}$$

$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 0.38\text{kNm}}{17.47\text{mm}} = 87.70\text{kN}$$

Model 2: Bolt failure with yielding of the end plate

$$M_{pl,2,Rd} = \frac{0.25 \cdot l_{eff,2} \cdot t_p^2 \cdot f_{yp}}{Y_{M0}} = \frac{0.25 \cdot 67.45\text{mm} \cdot (8.00\text{mm})^2 \cdot 355.00\text{MPa}}{1.000} = 0.38\text{kNm}$$

$$F_{T,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \cdot \sum F_{t,Rd}}{m_{ep} + n} = \frac{2 \cdot 0.38 kNm + 20.00 mm \cdot 2 \cdot 28.40 kN}{17.47 mm + 20.00 mm} = 50.76 kN$$

Model 3: Bolt failure

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 28.40 kN = 56.79 kN$$

Component resistance

$$F_{t,ep,Rd} = \min(F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}) = \min(87.70 kN; 50.76 kN; 56.79 kN) = 50.76 kN$$

Concrete cone failure

Actual area of concrete cone

$$A_{c,N} = (c_1 + s_1 + 0.5 \cdot s_{cr,N}) \cdot (c_2 + s_2 + 0.5 \cdot s_{cr,N}) = (80.00 mm + 0.00 mm + 0.5 \cdot 160.00 mm) \cdot (95.00 mm + 110.00 mm + 0.5 \cdot 160.00 mm) = 432.00 cm^2$$

Maximum area of concrete cone

$$A_{c,N}^0 = s_{cr,N} \cdot s_{cr,N} = 160.00 mm \cdot 160.00 mm = 256.00 cm^2$$

Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete

$$\psi_{s,N} = 0.7 + 0.3 \cdot \frac{c}{c_{cr,N}} = 0.7 + 0.3 \cdot \frac{80.00 mm}{80.00 mm} = 1.000$$

Shell spalling factor

$$\psi_{re,N} = 0.5 + \frac{h_{ef}}{200} = 0.5 + \frac{53.33 mm}{200} = 0.767$$

Factor related to distribution of tensile forces acting on anchors

1.000

Characteristic anchor resistance in non-cracked concrete

$$N_{Rk,c}^0 = k_1 \cdot \sqrt{f_{ck}} \cdot h_{ef}^{1.5} = 8.900 \cdot \sqrt{30.00 MPa} \cdot (53.33 mm)^{1.5} = 18.99 kN$$

$$N_{Rkc} = N_{Rk,c}^0 \cdot \frac{A_{cN}}{A_{0cN}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N} = 18.99 kN \cdot \frac{432.00 cm^2}{256.00 cm^2} \cdot 1.000 \cdot 0.767 \cdot 1.000 \cdot 1.000 = 24.56 kN$$

Design anchor resistance to concrete cone failure

$$N_{Rd,c} = \frac{N_{Rkc}}{\gamma_{MC}} = \frac{24.56 kN}{1.800} = 13.65 kN$$

Compression of concrete

Additional width of the bearing pressure zone

$$c = t_p \cdot \sqrt{\frac{f_{yp}}{3 \cdot f_{cd} \cdot \gamma_{M0}}} = 8.00 mm \cdot \sqrt{\frac{355.00 MPa}{3 \cdot 20.00 MPa \cdot 1.000}} = 19.46 mm$$

Bearing area for compression



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$$\begin{aligned}
 A_{eff} &= 2 \cdot (b_{fc} + 2 \cdot c_y) \cdot (c_z + t_{fc} + c) + (t_{wc} + 2 \cdot c) \cdot (h_c - 2 \cdot c - 2 \cdot t_{fc}) = \\
 &= 2 \cdot (100.00\text{mm} + 2 \cdot 5.54\text{mm}) \cdot (19.46\text{mm} + 8.00\text{mm} + 19.46\text{mm}) + (5.00\text{mm} + 2 \cdot 19.46\text{mm}) \cdot (96.00\text{mm} - 2 \cdot 19.46\text{mm} - 2 \cdot 5.54\text{mm}) = \\
 &= 122.28\text{cm}^2
 \end{aligned}$$

Bearing resistance of concrete for compression

$$F_{c,Rd} = A_{eff} \cdot f_{cd} = 122.28\text{cm}^2 \cdot 20.00\text{MPa} = 244.56\text{kN}$$

$ N_{Ed} \leq F_{c,Rd}$	$ -49.69\text{kN} < 244.56\text{kN}$	0.203	✓
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Bending resistance

Axial force eccentricity

$$e = \frac{M_{z,Ed}}{N_{Ed}} = \frac{-0.26\text{kNm}}{-49.69\text{kN}} = 5.15\text{mm}$$

Arm of compressive internal force

$$z_c = 0.5 \cdot (b_{fc} - t_{wc}) = 0.5 \cdot (100.00\text{mm} - 5.00\text{mm}) = 47.50\text{mm}$$

Arm of internal forces

$$z = 2 \cdot z_c = 2 \cdot 47.50\text{mm} = 95.00\text{mm}$$

Tensile resistance of an anchor

$$F_{c,Rd} = \min(F_{c,Rd}; F_{cf,Rd}) = \min(244.56\text{kN}; 334.90\text{kN}) = 244.56\text{kN}$$

$$M_{jRd1} = \left| \frac{-F_{c,Rd} \cdot z}{\frac{z_c}{e} + 1} \right| = \left| \frac{-244.56\text{kN} \cdot 95.00\text{mm}}{\frac{47.50\text{mm}}{5.15\text{mm}} + 1} \right| = 2.27\text{kNm}$$

$$M_{jRd2} = \left| \frac{-F_{c,Rd} \cdot z}{\frac{z_t}{e} - 1} \right| = \left| \frac{-244.56\text{kN} \cdot 95.00\text{mm}}{\frac{47.50\text{mm}}{5.15\text{mm}} - 1} \right| = 2.83\text{kNm}$$

$$M_{jRdz} = \min(M_{jRd1}; M_{jRd2}) = \min(2.27\text{kNm}; 2.83\text{kNm}) = 2.27\text{kNm}$$

$ M_{z,Ed} \leq M_{jRdz}$	$ -0.26\text{kNm} < 2.27\text{kNm}$	0.113	✓
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Shear

Shear section area of bolt

$$A = 0.25 \cdot \pi \cdot d^2 = 0.25 \cdot 3.142 \cdot (10.00\text{mm})^2 = 0.79\text{cm}^2$$

Shear resistance of bolt in one surface



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$$F_{v,Rd} = \frac{\alpha_v \cdot m \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0.600 \cdot 1 \cdot 800.00 \text{ MPa} \cdot 0.79 \text{ cm}^2}{1.250} = 30.16 \text{ kN}$$

Coefficient

$$\alpha_b = 0.44 - 0.0003 \cdot f_{yb} = 0.44 - 0.0003 \cdot 640.00 \text{ MPa} = 0.248$$

Shear of an anchor bolt

$$F_{2,vb,Rd} = \frac{\alpha_b \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.248 \cdot 800.00 \text{ MPa} \cdot 0.58 \text{ cm}^2}{1.250} = 9.21 \text{ kN}$$

Coefficient determined by bolt spacing

$$\alpha_{ep} = \min(1.0; \frac{f_{ub}}{f_{up}}; \frac{e_1}{3 \cdot d_0}) = \min(1.0; \frac{800.00 \text{ MPa}}{490.00 \text{ MPa}}; \frac{20.00 \text{ mm}}{3 \cdot 11.00 \text{ mm}}) = 0.606$$

Coefficient determined by bolt spacing

$$k_1 = \min(2.5; 2.8 \cdot \frac{e_2}{d_0} - 1.7) = \min(2.5; 2.8 \cdot \frac{20.00 \text{ mm}}{11.00 \text{ mm}} - 1.7) = 2.500$$

Bearing resistance of a bolt

$$F_{b,Rd} = k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p = 2.500 \cdot 0.606 \cdot 490.00 \text{ MPa} \cdot 10.00 \text{ mm} \cdot 8.00 \text{ mm} = 77.58 \text{ kN}$$

Resistance of the bolt row

$$V_{j,Rdy} = 4 \cdot \min(77.58 \text{ kN}; 30.16 \text{ kN}; 9.21 \text{ kN}; 5.05 \text{ kN}; 13.65 \text{ kN}) = 20.19 \text{ kN}$$

$\frac{ V_{y,Ed} }{V_{j,Rdy}} \leq 1$	$ 0.13 \text{ kN} < 20.19 \text{ kN}$	0.006	✓
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Fillet welds connecting beam and front plate

Geometric properties of welds

Area of horizontal welds on upper flange

$$A_{wfu} = [b_{fc} + (b_{fc} - t_{wc} - 2 \cdot r_c)] \cdot a_{fc} = [100.00 \text{ mm} + (100.00 \text{ mm} - 5.00 \text{ mm} - 2 \cdot 12.00 \text{ mm})] \cdot 4.00 \text{ mm} = 6.84 \text{ cm}^2$$

Area of horizontal welds on lower flange

$$A_{wfl} = [b_{fc} + (b_{fc} - t_{wc} - 2 \cdot r_c)] \cdot a_{fc} = [100.00 \text{ mm} + (100.00 \text{ mm} - 5.00 \text{ mm} - 2 \cdot 12.00 \text{ mm})] \cdot 4.00 \text{ mm} = 6.84 \text{ cm}^2$$

Area of vertical welds

$$A_{ww} = 2 \cdot (h_c - 2 \cdot (t_{fc} + r_c)) \cdot a_{wc} = 2 \cdot (96.00 \text{ mm} - 2 \cdot (8.00 \text{ mm} + 12.00 \text{ mm})) \cdot 4.00 \text{ mm} = 4.48 \text{ cm}^2$$

Area of all welds

$$A_w = A_{wfu} + A_{wfl} + A_{ww} = 6.84 \text{ cm}^2 + 6.84 \text{ cm}^2 + 4.48 \text{ cm}^2 = 18.16 \text{ cm}^2$$

Offset of welds centroid with respect to beam centroid

$$e_{0w} = 0.00 \text{ mm}$$

Moment of inertia welds

$$I_w = 293.91 \text{ cm}^4$$



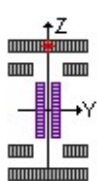
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The point where the stresses are checked	$z_i = 50.00mm$
Elastic modulus of welds	
$W_w = 58.78cm^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.69kN}{18.16cm^2} = -27.36MPa$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{W_w} = \frac{0.00kNm \cdot 50.00mm}{293.91cm^4} = 0.00MPa$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -27.36MPa + 0.00MPa = -27.36MPa$	
Normal perpendicular stress	
$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-27.36MPa}{\sqrt{2}} = -19.35MPa$	
Perpendicular tangent stress	
$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-27.36MPa}{\sqrt{2}} = -19.35MPa$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -19.35MPa < 352.80MPa$	0.055	✓
---	---------------------------	-------	---

$\sqrt{\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$38.70MPa < 435.56MPa$	0.089	✓
---	------------------------	-------	---

The point where the stresses are checked	$z_i = 28.00mm$
--	-----------------



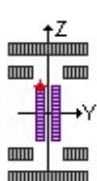
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Elastic modulus of welds	
$W_w = 104.97cm^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.69kN}{18.16cm^2} = -27.36MPa$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{W_w} = \frac{0.00kNm \cdot 28.00mm}{293.91cm^4} = 0.00MPa$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -27.36MPa + 0.00MPa = -27.36MPa$	
Normal perpendicular stress	
$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-27.36MPa}{\sqrt{2}} = -19.35MPa$	
Perpendicular tangent stress	
$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-27.36MPa}{\sqrt{2}} = -19.35MPa$	
Parallel tangent stress	
$\tau_{ } = \frac{V_0}{A_{ww}} = \frac{0.00kN}{18.16cm^2} = 0.00MPa$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -19.35MPa < 352.80MPa$	0.055	✓
---	---------------------------	-------	---

$\sqrt{\sigma_{\perp}^2 + 3 \cdot \tau_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$38.70MPa < 435.56MPa$	0.089	✓
---	------------------------	-------	---

The point where the stresses are checked	$z_i = -28.00mm$
--	------------------



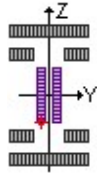
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Elastic modulus of welds	
$W_w = 104.97cm^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.69kN}{18.16cm^2} = -27.36MPa$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{W_w} = \frac{0.00kNm \cdot (-28.00mm)}{293.91cm^4} = -0.00MPa$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -27.36MPa + -0.00MPa = -27.36MPa$	
Normal perpendicular stress	
$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-27.36MPa}{\sqrt{2}} = -19.35MPa$	
Perpendicular tangent stress	
$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-27.36MPa}{\sqrt{2}} = -19.35MPa$	
Parallel tangent stress	
$\tau_{ } = \frac{V_0}{A_{ww}} = \frac{0.00kN}{18.16cm^2} = 0.00MPa$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -19.35MPa < 352.80MPa$	0.055	✓
---	---------------------------	-------	---

$\sqrt{\sigma_{\perp}^2 + 3 \cdot \tau_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$38.70MPa < 435.56MPa$	0.089	✓
---	------------------------	-------	---

The point where the stresses are checked	$z_i = -50.00mm$
--	------------------



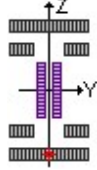
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Elastic modulus of welds	
$W_w = 58.78 \text{ cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.69 \text{ kN}}{18.16 \text{ cm}^2} = -27.36 \text{ MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{W_w} = \frac{0.00 \text{ kNm} \cdot (-50.00 \text{ mm})}{293.91 \text{ cm}^4} = -0.00 \text{ MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -27.36 \text{ MPa} + -0.00 \text{ MPa} = -27.36 \text{ MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-27.36 \text{ MPa}}{\sqrt{2}} = -19.35 \text{ MPa}$	
Perpendicular tangent stress	
$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-27.36 \text{ MPa}}{\sqrt{2}} = -19.35 \text{ MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -19.35 \text{ MPa} < 352.80 \text{ MPa}$	0.055	✓
---	---	-------	---

$\sqrt{\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$38.70 \text{ MPa} < 435.56 \text{ MPa}$	0.089	✓
---	--	-------	---

The rotational stiffness of a joint

The bolt elongation length

$$L_b = 8 \cdot d + t_p + t_{wa} + 0.5 \cdot m = 8 \cdot 10.00 \text{ mm} + 8.00 \text{ mm} + 2.00 \text{ mm} + 0.5 \cdot 9.50 \text{ mm} = 114.75 \text{ mm}$$

Bolt tension

$$k_{13} = \frac{E_c \cdot \sqrt{b_{eff} \cdot l_{eff}}}{1.275 \cdot E} = \frac{32800.00 \text{ MPa} \cdot \sqrt{0.00 \text{ mm} \cdot 0.00 \text{ mm}}}{1.275 \cdot 210000.00 \text{ MPa}} = 0.00 \text{ mm}$$

End plate in bending

$$k_{15} = \frac{0.425 \cdot l_{eff} \cdot t_p^3}{m_x^3} = \frac{0.425 \cdot 67.45 \text{ mm} \cdot (8.00 \text{ mm})^3}{(17.47 \text{ mm})^3} = 2.75 \text{ mm}$$

Bolt tension



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$$k_{16} = \frac{2 \cdot A_b}{L_b} = \frac{1.6 \cdot 0.58 \text{ cm}^2}{114.75 \text{ mm}} = 1.01 \text{ mm}$$

$$k_t = k_{15} + k_{16} = 2.75 \text{ mm} + 1.01 \text{ mm} = 3.76 \text{ mm}$$

$$k_c = k_{13} = 0.00 \text{ mm}$$

The initial rotational stiffness of a joint

$$S_{j,ini} = \frac{E \cdot z^2}{\frac{1}{k_c} + \frac{1}{k_t}} \cdot \frac{e}{e + e_k} = \frac{210000.00 \text{ MPa} \cdot (95.00 \text{ mm})^2}{\frac{1}{0.00 \text{ mm}} + \frac{1}{3.76 \text{ mm}}} \cdot \frac{5.15 \text{ mm}}{5.15 \text{ mm} + \text{NaN mm}} = \text{NaN} \frac{\text{kNm}}{\text{rad}}$$

$$S_{j,ini} = \frac{E \cdot z^2}{\frac{1}{k_c} + \frac{1}{k_t}} \cdot \frac{e}{e + e_k} = \frac{210000.00 \text{ MPa} \cdot (95.00 \text{ mm})^2}{\frac{1}{0.00 \text{ mm}} + \frac{1}{3.76 \text{ mm}}} \cdot \frac{5.15 \text{ mm}}{5.15 \text{ mm} + \text{NaN mm}} = \text{NaN} \frac{\text{kNm}}{\text{rad}}$$

The rotational stiffness of a pinned joint

$$S_{j,pin} = \frac{0.5 \cdot E \cdot I_{yc}}{L_c} = \frac{0.5 \cdot 210000.00 \text{ MPa} \cdot 349.24 \text{ cm}^4}{2520.00 \text{ mm}} = 145.52 \frac{\text{kNm}}{\text{rad}}$$

The rotational stiffness of a rigid joint

$$S_{j,rig} = \frac{30 \cdot E \cdot I_{yc}}{L_c} = \frac{30 \cdot 210000.00 \text{ MPa} \cdot 349.24 \text{ cm}^4}{2520.00 \text{ mm}} = 8730.91 \frac{\text{kNm}}{\text{rad}}$$

Stiffness ratio

$$\mu = 1.000$$

The rotational stiffness of a joint

$$S_j = \frac{S_{j,ini}}{\mu} = \frac{\text{NaN} \frac{\text{kNm}}{\text{rad}}}{1.000} = \text{NaN} \frac{\text{kNm}}{\text{rad}}$$

Classification of joint

Rigid



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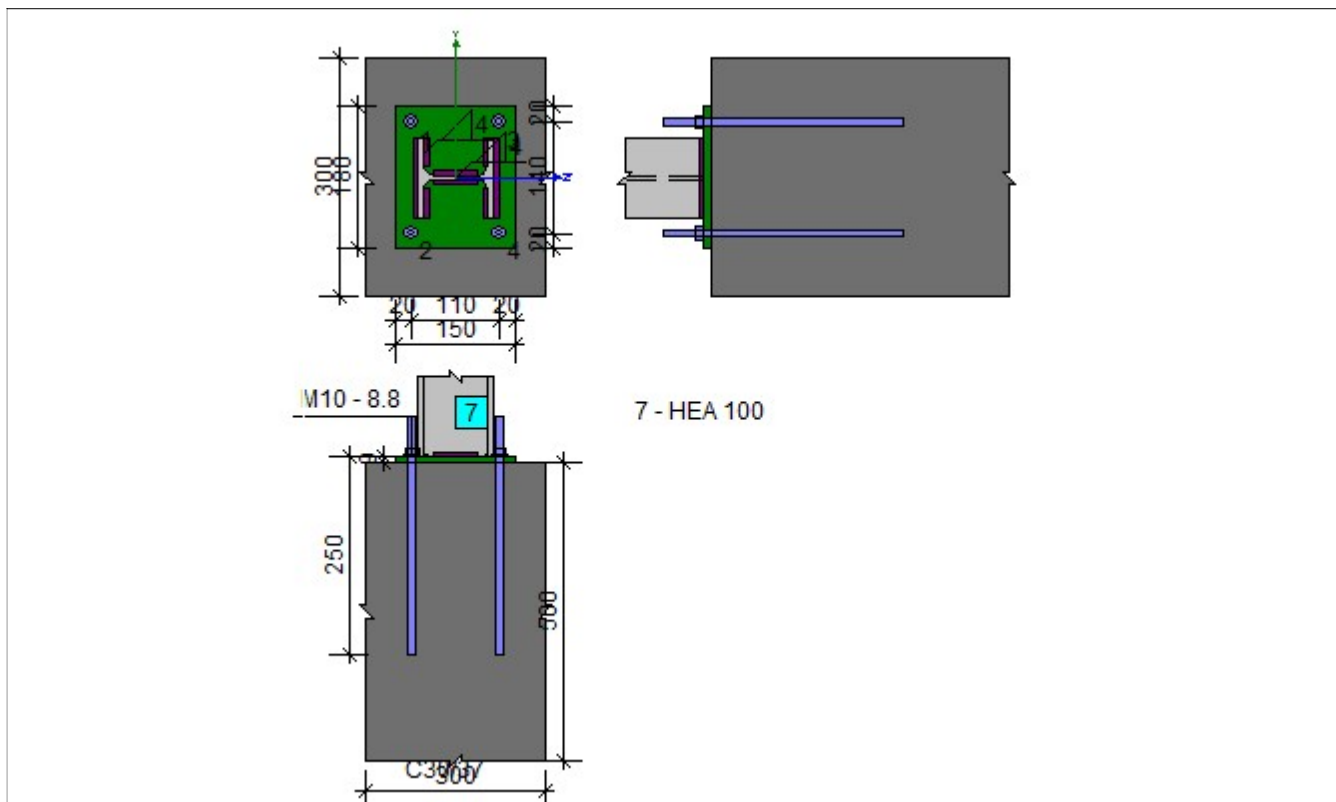
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	Column base EC3 1991-1-8: 2008, EN 1992-4:2018	Ratio : 0.962	
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Structure node: 18

Structure members: 7



Data

Column HEA 100					
	h_c	b_{fc}	t_{fc}	t_{wc}	R_c
	96.00mm	100.00mm	8.00mm	5.00mm	12.00mm
	A_c	J_{y0c}	J_{z0c}	y_{0c}	z_{0c}
	21.24cm ²	349.24cm ⁴	133.81cm ⁴	50.00mm	48.00mm
Material	Grade	f_y	f_u		
	S 355	355.00MPa	490.00MPa		

Base plate			
	l_p	h_p	t_p
	150.00mm	180.00mm	8.50mm
Material	Grade	f_y	f_u
	S 355	355.00MPa	490.00MPa



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Anchorage

Grade		8.8
Yield point	$f_y =$	640.00MPa
Resistance on tension	$f_u =$	800.00MPa
Bolt diameter	$d =$	10.00mm
Bolt opening diameter	$d_0 =$	11.00mm
Area of bolt section	$A =$	0.79cm ²
Effective section area of a bolt	$A_s =$	0.58cm ²
Number of rows	$n_y =$	2.00
Number of rows	$n_z =$	2.00
Vertical spacing of rows		$p_{y1} = 140.00mm$
Vertical spacing of rows		$p_{z1} = 110.00mm$

Welds

Thickness of fillet welds connecting beam flanges and front plate	$a_{fc} =$	4.00mm
Thickness of fillet welds connecting beam web and front plate	$a_{wc} =$	4.00mm

Foundation			
	L_{1f}	B_{1f}	H_{1f}
	300.00mm	300.00mm	500.00mm
Material	Grade	E_c	f_{ck}
	C30/37	32800.00MPa	30.00MPa

Materials coeffs

Coefficient	$\gamma_{M0} =$	1.000
Coefficient	$\gamma_{M2} =$	1.250
Coefficient	$\gamma_C =$	1.500
Coefficient	$\gamma_{Inst} =$	1.200

Forces

Design loads

Design loads [1.35*0.85*Gk] {1.5*Qk}



Magnētiskās rezonanses iekārtas uzstādīšana Slimnīcas ielā 25, Lie

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Axial force	$N_{Ed} =$	$-76.29kN$	
Shear force	$V_{y,Ed} =$	$-0.06kN$	
Shear force	$V_{z,Ed} =$	$6.55kN$	
Bending moment	$M_{y,Ed} =$	$-5.06kNm$	
Bending moment	$M_{z,Ed} =$	$-0.24kNm$	

In this version only the most external anchor row on tension side is taking into account for bending moment resistance of joint

Results

Beam flange and web in compression

Plastic section modulus

$$W_{pl} = 83.02cm^3$$

The design resistance for bending of cross-section

$$M_{c,Rd} = \frac{W_{pl} \cdot f_{yb}}{\gamma_{M0}} = \frac{83.02cm^3 \cdot 355.00MPa}{1.000} = 29.47kNm$$

Distance between beam flanges

$$h_f = h_c - t_{fc} = 96.00mm - 8.00mm = 88.00mm$$

Design resistance of column web subject to transverse compression

$$F_{c,fc,Rd} = \frac{M_{c,Rd}}{h_f} = \frac{29.47kNm}{88.00mm} = 334.90kN$$

Resistance of a column base in the tension zone

Tensile resistance of a bolt

$$F_{t,Rd} = \frac{0.85 \cdot k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.85 \cdot 0.900 \cdot 800.00MPa \cdot 0.58cm^2}{1.250} = 28.40kN$$

Pull-out failure

Design tensile resistance

$$f_{ctd} = \frac{0.7 \cdot 0.3 \cdot f_{ck}^{\frac{2}{3}}}{\gamma_c} = \frac{0.7 \cdot 0.3 \cdot (30.00MPa)^{\frac{2}{3}}}{1.500} = 1.35MPa$$



Coefficient related to the quality of the bond conditions and concreting conditions

$$\eta_1 = 1.000$$

Coefficient related to the member diameter

$$\eta_2 = 1.000$$

Design value of the ultimate bond stress

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 2.25 \cdot 1.000 \cdot 1.000 \cdot 1.35 \text{ MPa} = 3.04 \text{ MPa}$$

Design anchorage length

$$L_{eff} = 250.00 \text{ mm}$$

Design uplift capacity

$$F_{btd} = \pi \cdot d \cdot L_{eff} \cdot f_{bd} = \pi \cdot 10.00 \text{ mm} \cdot 250.00 \text{ mm} \cdot 3.04 \text{ MPa} = 23.89 \text{ kN}$$

Geometrical parameters

Bolt distance from outer edge

$$e_{ep} = 20.00 \text{ mm}$$

Bolt distance from beam flange

$$m_x = 0.5 \cdot (p_{1z} - h_c) - 0.8 \cdot \sqrt{2} \cdot a_{fc} = 0.5 \cdot (110.00 \text{ mm} - 96.00 \text{ mm}) - 0.8 \cdot \sqrt{2} \cdot 4.00 \text{ mm} = 2.47 \text{ mm}$$

Bolt distance from outer horizontal edge plate

$$e_x = e_1 = 20.00 \text{ mm}$$

Effective length for one bolt in circular patterns

$$l_{eff,cp,1} = 2 \cdot \pi \cdot m_x = 2 \cdot \pi \cdot 2.47 \text{ mm} = 15.55 \text{ mm}$$

$$l_{eff,cp,2} = \pi \cdot m_x + 2 \cdot e = \pi \cdot 2.47 \text{ mm} + 2 \cdot 20.00 \text{ mm} = 47.77 \text{ mm}$$

$$l_{eff,cp,3} = \pi \cdot m_x + w = \pi \cdot 2.47 \text{ mm} + 140.00 \text{ mm} = 147.77 \text{ mm}$$

$$l_{eff,cp} = \min(l_{eff,cp,1}; l_{eff,cp,2}; l_{eff,cp,3}) = \min(15.55 \text{ mm}; 47.77 \text{ mm}; 147.77 \text{ mm}) = 15.55 \text{ mm}$$

Effective length for one bolt in non-circular patterns

$$l_{eff,nc,1} = 4 \cdot m_x + 1.25 \cdot e_x = 4 \cdot 2.47 \text{ mm} + 1.25 \cdot 20.00 \text{ mm} = 34.90 \text{ mm}$$

$$l_{eff,nc,2} = e + 2 \cdot m_x + 0.625 \cdot e_x = 20.00 \text{ mm} + 2 \cdot 2.47 \text{ mm} + 0.625 \cdot 20.00 \text{ mm} = 37.45 \text{ mm}$$

$$l_{eff,nc,3} = 0.5 \cdot b_p = 0.5 \cdot 180.00 \text{ mm} = 90.00 \text{ mm}$$

$$l_{eff,nc,4} = 0.5 \cdot w + 2 \cdot m_x + 0.625 \cdot e_x = 0.5 \cdot 140.00 \text{ mm} + 2 \cdot 2.47 \text{ mm} + 0.625 \cdot 20.00 \text{ mm} = 87.45 \text{ mm}$$

$$l_{eff,nc} = \min(l_{eff,nc,1}; l_{eff,nc,2}; l_{eff,nc,3}; l_{eff,nc,4}) = \min(34.90 \text{ mm}; 37.45 \text{ mm}; 90.00 \text{ mm}; 87.45 \text{ mm}) = 34.90 \text{ mm}$$

Effective length for one bolt for 1 mode

$$l_{eff,1} = \min(l_{eff,cp}; l_{eff,nc}) = \min(15.55 \text{ mm}; 34.90 \text{ mm}) = 15.55 \text{ mm}$$



Effective length for one bolt for 2 mode

$$l_{eff,2} = l_{eff,nc} = 34.90mm$$

Model 1: Complete yielding of the end plate

$$M_{pl,1,Rd} = \frac{0.25 \cdot l_{eff,1} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 15.55mm \cdot (8.50mm)^2 \cdot 355.00MPa}{1.000} = 0.10kNm$$

$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 0.10kNm}{2.47mm} = 161.16kN$$

Model 2: Bolt failure with yielding of the end plate

$$M_{pl,2,Rd} = \frac{0.25 \cdot l_{eff,2} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 34.90mm \cdot (8.50mm)^2 \cdot 355.00MPa}{1.000} = 0.22kNm$$

$$F_{T,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \cdot \sum F_{t,Rd}}{m_{ep} + n} = \frac{2 \cdot 0.22kNm + 3.09mm \cdot 2 \cdot 28.40kN}{2.47mm + 3.09mm} = 111.94kN$$

Model 3: Bolt failure

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 28.40kN = 56.79kN$$

Component resistance

$$F_{t,ep,Rd} = \min(F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}) = \min(161.16kN; 111.94kN; 56.79kN) = 56.79kN$$

Concrete cone failure

Actual area of concrete cone

$$A_{c,N} = (c_1 + s_1 + 0.5 \cdot s_{cr,N}) \cdot (c_2 + s_2 + 0.5 \cdot s_{cr,N}) = \\ = (95.00mm + 0.00mm + 0.5 \cdot 160.00mm) \cdot (80.00mm + 140.00mm + 0.5 \cdot 160.00mm) = 480.00cm^2$$

Maximum area of concrete cone

$$A_{c,N}^0 = s_{cr,N} \cdot s_{cr,N} = 160.00mm \cdot 160.00mm = 256.00cm^2$$

Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete

$$\psi_{s,N} = 0.7 + 0.3 \cdot \frac{c}{c_{cr,N}} = 0.7 + 0.3 \cdot \frac{80.00mm}{80.00mm} = 1.000$$

Shell spalling factor

$$\psi_{re,N} = 0.5 + \frac{h_{ef}}{200} = 0.5 + \frac{53.33mm}{200} = 0.767$$



Factor related to distribution of tensile forces acting on anchors

1.000

Characteristic anchor resistance in non-cracked concrete

$$N_{Rk,c}^0 = k_1 \cdot \sqrt{f_{ck}} \cdot h_{ef}^{1.5} = 8.900 \cdot \sqrt{30.00 \text{ MPa}} \cdot (53.33 \text{ mm})^{1.5} = 18.99 \text{ kN}$$

$$N_{Rk,c} = N_{Rk,c}^0 \cdot \frac{A_{cN}}{A_{0cN}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N} = 18.99 \text{ kN} \cdot \frac{480.00 \text{ cm}^2}{256.00 \text{ cm}^2} \cdot 1.000 \cdot 0.767 \cdot 1.000 \cdot 1.000 = 27.29 \text{ kN}$$

Design anchor resistance to concrete cone failure

$$N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{MC}} = \frac{27.29 \text{ kN}}{1.800} = 15.16 \text{ kN}$$

Compression of concrete

Additional width of the bearing pressure zone

$$c = t_p \cdot \sqrt{\frac{f_{yp}}{3 \cdot f_{cd} \cdot \gamma_{M0}}} = 8.50 \text{ mm} \cdot \sqrt{\frac{355.00 \text{ MPa}}{3 \cdot 20.00 \text{ MPa} \cdot 1.000}} = 20.68 \text{ mm}$$

Effective width of the bearing pressure zone under the flange

$$b_{eff} = 2 \cdot c + t_{fc} = 2 \cdot 20.68 \text{ mm} + 8.00 \text{ mm} = 49.35 \text{ mm}$$

Effective length of the bearing pressure zone under the flange

$$l_{eff} = \min(b_{p'}; 2 \cdot c + b_{fc}) = \min(180.00 \text{ mm}; 2 \cdot 20.68 \text{ mm} + 100.00 \text{ mm}) = 141.35 \text{ mm}$$

Bearing area for bending M_y

$$A_{eff} = b_{eff} \cdot l_{eff} = 49.35 \text{ mm} \cdot 141.35 \text{ mm} = 69.76 \text{ cm}^2$$

Bearing resistance of concrete for compression

$$F_{c,Rd} = A_{eff} \cdot f_{cd} = 69.76 \text{ cm}^2 \cdot 20.00 \text{ MPa} = 139.52 \text{ kN}$$

$ N_{Ed} \leq F_{c,Rd}$	$ - 76.29 \text{ kN} < 139.52 \text{ kN}$	0.547	✓
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Bending resistance

Axial force eccentricity

$$e = \frac{M_{y,Ed}}{N_{Ed}} = \frac{- 5.06 \text{ kNm}}{- 76.29 \text{ kN}} = 66.35 \text{ mm}$$

Arm of compressive internal force

$$z_c = 0.5 \cdot (h_c - t_{fc}) = 0.5 \cdot (96.00 \text{ mm} - 8.00 \text{ mm}) = 44.00 \text{ mm}$$

Arm of tensile internal force

$$z_t = 55.00 \text{ mm}$$

Arm of internal forces

$$z = z_t + z_c = 55.00 \text{ mm} + 44.00 \text{ mm} = 99.00 \text{ mm}$$

$$F_{c,Rd} = \min(F_{c,Rd'}; F_{cf,c,Rd}) = \min(139.52 \text{ kN}; 334.90 \text{ kN}) = 139.52 \text{ kN}$$

Tensile resistance of an anchor

$$F_{T,Rd} = \min(F_{t,ep,Rd'}; F_{btd} \cdot n_{a'}; N_{Rdc} \cdot n_a) = \min(56.79 \text{ kN}; 23.89 \text{ kN} \cdot 2; 15.16 \text{ kN} \cdot 2) = 30.33 \text{ kN}$$



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$$M_{jRd1} = \left| \frac{-F_{C,Rd} \cdot z}{\frac{z_t}{e} + 1} \right| = \left| \frac{-139.52kN \cdot 99.00mm}{\frac{55.00mm}{66.35mm} + 1} \right| = 7.55kNm$$

$$M_{jRd2} = \left| \frac{F_{T,Rd} \cdot z}{\frac{z_c}{e} - 1} \right| = \left| \frac{30.33kN \cdot 99.00mm}{\frac{44.00mm}{66.35mm} - 1} \right| = 8.91kNm$$

$$M_{jRdy} = \min(M_{jRd1}; M_{jRd2}) = \min(7.55kNm; 8.91kNm) = 7.55kNm$$

$ M_{y,Ed} \leq M_{jRdy}$	$ - 5.06kNm < 7.55kNm$	0.670	✓
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Bending resistance

Axial force eccentricity

$$e = \frac{M_{z,Ed}}{N_{Ed}} = \frac{-0.24kNm}{-76.29kN} = 3.13mm$$

Arm of compressive internal force

$$z_c = 0.5 \cdot (b_{fc} - t_{wc}) = 0.5 \cdot (100.00mm - 5.00mm) = 47.50mm$$

Arm of internal forces

$$z = 2 \cdot z_c = 2 \cdot 47.50mm = 95.00mm$$

Tensile resistance of an anchor

$$F_{C,Rd} = \min(F_{C,Rd}; F_{cf,Rd}) = \min(139.52kN; 334.90kN) = 139.52kN$$

$$M_{jRd1} = \left| \frac{-F_{C,Rd} \cdot z}{\frac{z_c}{e} + 1} \right| = \left| \frac{-139.52kN \cdot 95.00mm}{\frac{47.50mm}{3.13mm} + 1} \right| = 0.82kNm$$

$$M_{jRd2} = \left| \frac{-F_{C,Rd} \cdot z}{\frac{z_t}{e} - 1} \right| = \left| \frac{-139.52kN \cdot 95.00mm}{\frac{47.50mm}{3.13mm} - 1} \right| = 0.94kNm$$

$$M_{jRdz} = \min(M_{jRd1}; M_{jRd2}) = \min(0.82kNm; 0.94kNm) = 0.82kNm$$

$ M_{z,Ed} \leq M_{jRdz}$	$ - 0.24kNm < 0.82kNm$	0.291	✓
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$$\frac{|M_{y,Ed}|}{M_{j,Rdy}} + \frac{|M_{z,Ed}|}{M_{j,Rdz}} = \frac{|-5.06kNm|}{7.55kNm} + \frac{|-0.24kNm|}{0.82kNm} = 0.962$$

$\frac{ M_{y,Ed} }{M_{j,Rdy}} + \frac{ M_{z,Ed} }{M_{j,Rdz}}$	0.962 < 1.000	0.962	✓
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Shear

Shear section area of bolt

$$A = 0.25 \cdot \pi \cdot d^2 = 0.25 \cdot 3.142 \cdot (10.00mm)^2 = 0.79cm^2$$

Shear resistance of bolt in one surface

$$F_{v,Rd} = \frac{\alpha_v \cdot m \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0.600 \cdot 1 \cdot 800.00MPa \cdot 0.79cm^2}{1.250} = 30.16kN$$

Coefficient

$$\alpha_b = 0.44 - 0.0003 \cdot f_{yb} = 0.44 - 0.0003 \cdot 640.00MPa = 0.248$$

Shear of an anchor bolt

$$F_{2,vb,Rd} = \frac{\alpha_b \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.248 \cdot 800.00MPa \cdot 0.58cm^2}{1.250} = 9.21kN$$

Coefficient determined by bolt spacing

$$\alpha_{ep} = \min(1.0; \frac{f_{ub}}{f_{up}}; \frac{e_1}{3 \cdot d_0}) = \min(1.0; \frac{800.00MPa}{490.00MPa}; \frac{20.00mm}{3 \cdot 11.00mm}) = 0.606$$

Coefficient determined by bolt spacing

$$k_1 = \min(2.5; 2.8 \cdot \frac{e_2}{d_0} - 1.7) = \min(2.5; 2.8 \cdot \frac{20.00mm}{11.00mm} - 1.7) = 2.500$$

Bearing resistance of a bolt

$$F_{b,Rd} = k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p = 2.500 \cdot 0.606 \cdot 490.00MPa \cdot 10.00mm \cdot 8.50mm = 82.42kN$$

Resistance of the bolt row

$$V_{j,Rdy} = 4 \cdot \min(82.42kN; 30.16kN; 9.21kN; 5.05kN; 15.16kN) = 20.19kN$$

$\frac{ V_{y,Ed} }{V_{j,Rdy}} \leq 1$	$ 0.06kN < 20.19kN$	0.003	✓
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Shear resistance of a bolt - with lever arm - Not analyzed !

Shear resistance of a bolt - without lever arm



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$$V_{Rk,s}^0 = k_6 \cdot A_s \cdot f_{uk} = 0.600 \cdot 0.79 \text{ cm}^2 \cdot 800.00 \text{ MPa} = 37.70 \text{ kN}$$

Concrete pry-out failure

$$V_{Rk,cp} = k_8 \cdot N_{Rkc} = 1.000 \cdot 27.29 \text{ kN} = 27.29 \text{ kN}$$

$$V_{Rd,cp} = \frac{V_{Rk,c}}{\gamma_{MC}} = \frac{27.29 \text{ kN}}{1.800} = 15.16 \text{ kN}$$

Concrete edge failure

Factor related to the influence of edges parallel to the shear load direction

$$\psi_{eV} = \frac{0.7 + 0.3 \cdot c_2}{1.5 \cdot c_1} = \frac{0.7 + 0.3 \cdot 80.00 \text{ mm}}{1.5 \cdot 80.00 \text{ mm}} = 0.900$$

Factor related to the foundation thickness

$$\psi_{hV} = \left(\frac{1.5 \cdot c_1}{h} \right)^{0.5} = \left(\frac{1.5 \cdot 80.00 \text{ mm}}{500.00 \text{ mm}} \right)^{0.5} = 1.000$$

Factor taking account a group effect when different shear loads are acting on the individual anchors in a group

$$\psi_{ecV} = \frac{1}{1 + 2 \cdot e_V} = \frac{1}{1 + 2 \cdot 0.00 \text{ mm}} = 1.000$$

Factor related to the angle at which the shear load is applied

$$\psi_{\alpha,V} = 1.000$$

Concrete resistance for edge failure

$$V_{Rk,c}^0 = k_9 \cdot d_{nom}^\alpha \cdot l_f^\beta \cdot \sqrt{f_{ck}} \cdot c_1^{1.5} = 1.700 \cdot (10.00 \text{ mm})^{0.122} \cdot (120.00 \text{ mm})^{0.066} \cdot \sqrt{30.00 \text{ MPa}} \cdot (80.00 \text{ mm})^{1.5} = 12.11 \text{ kN}$$

$$V_{Rk,c} = V_{Rk,c}^0 \cdot \frac{A_{c,V}}{A_{c,V}^0} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{ec,V} \cdot \psi_{\alpha,V} \cdot \psi_{re,V} = 12.11 \text{ kN} \cdot \frac{240.00 \text{ cm}^2}{288.00 \text{ cm}^2} \cdot 0.900 \cdot 1.000 \cdot 1.000 \cdot 1.000 \cdot 1.000 = 9.09 \text{ kN}$$

$$V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{MC}} = \frac{9.09 \text{ kN}}{1.800} = 5.05 \text{ kN}$$

Shear

Shear section area of bolt

$$A = 0.25 \cdot \pi \cdot d^2 = 0.25 \cdot 3.142 \cdot (10.00 \text{ mm})^2 = 0.79 \text{ cm}^2$$

Shear resistance of bolt in one surface

$$F_{v,Rd} = \frac{\alpha_v \cdot m \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0.600 \cdot 1 \cdot 800.00 \text{ MPa} \cdot 0.79 \text{ cm}^2}{1.250} = 30.16 \text{ kN}$$

Coefficient

$$\alpha_b = 0.44 - 0.0003 \cdot f_{yb} = 0.44 - 0.0003 \cdot 640.00 \text{ MPa} = 0.248$$

Shear of an anchor bolt

$$F_{2,vb,Rd} = \frac{\alpha_b \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.248 \cdot 800.00 \text{ MPa} \cdot 0.58 \text{ cm}^2}{1.250} = 9.21 \text{ kN}$$



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Coefficient determined by bolt spacing

$$\alpha_{ep} = \min\left(1.0; \frac{f_{ub}}{f_{up}}; \frac{e_1}{3 \cdot d_0}\right) = \min\left(1.0; \frac{800.00MPa}{490.00MPa}; \frac{20.00mm}{3 \cdot 11.00mm}\right) = 0.606$$

Coefficient determined by bolt spacing

$$k_1 = \min\left(2.5; 2.8 \cdot \frac{e_2}{d_0} - 1.7\right) = \min\left(2.5; 2.8 \cdot \frac{20.00mm}{11.00mm} - 1.7\right) = 2.500$$

Bearing resistance of a bolt

$$F_{b,Rd} = k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p = 2.500 \cdot 0.606 \cdot 490.00MPa \cdot 10.00mm \cdot 8.50mm = 82.42kN$$

Resistance of the bolt row

$$V_{j,Rdz} = 4 \cdot \min(82.42kN; 30.16kN; 9.21kN; 5.05kN; 15.16kN) = 20.19 \text{ kN}$$

$\frac{ V_{z,Ed} }{V_{j,Rdz}} \leq 1$	$ 6.55kN < 20.19kN$	0.324	✓
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Fillet welds connecting beam and front plate

Geometric properties of welds

Area of horizontal welds on upper flange

$$A_{wfu} = [b_{fc} + (b_{fc} - t_{wc} - 2 \cdot r_c)] \cdot a_{fc} = [100.00mm + (100.00mm - 5.00mm - 2 \cdot 12.00mm)] \cdot 4.00mm = 6.84cm^2$$

Area of horizontal welds on lower flange

$$A_{wfl} = [b_{fc} + (b_{fc} - t_{wc} - 2 \cdot r_c)] \cdot a_{fc} = [100.00mm + (100.00mm - 5.00mm - 2 \cdot 12.00mm)] \cdot 4.00mm = 6.84cm^2$$

Area of vertical welds

$$A_{ww} = 2 \cdot (h_c - 2 \cdot (t_{fc} + r_c)) \cdot a_{wc} = 2 \cdot (96.00mm - 2 \cdot (8.00mm + 12.00mm)) \cdot 4.00mm = 4.48cm^2$$

Area of all welds

$$A_w = A_{wfu} + A_{wfl} + A_{ww} = 6.84cm^2 + 6.84cm^2 + 4.48cm^2 = 18.16cm^2$$

Offset of welds centroid with respect to beam centroid

$$e_{0w} = 0.00mm$$

Moment of inertia welds

$$I_w = 293.91cm^4$$

The point where the stresses are checked	$z_i = 50.00mm$
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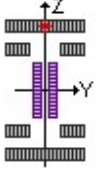
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Elastic modulus of welds	
$W_w = 58.78 \text{ cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-76.29 \text{ kN}}{18.16 \text{ cm}^2} = -42.01 \text{ MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{W_w} = \frac{-5.06 \text{ kNm} \cdot 50.00 \text{ mm}}{293.91 \text{ cm}^4} = -86.11 \text{ MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -42.01 \text{ MPa} + (-86.11 \text{ MPa}) = -128.12 \text{ MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-128.12 \text{ MPa}}{\sqrt{2}} = -90.60 \text{ MPa}$	
Perpendicular tangent stress	
$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-128.12 \text{ MPa}}{\sqrt{2}} = -90.60 \text{ MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -90.60 \text{ MPa} < 352.80 \text{ MPa}$	0.257	✓
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$\sqrt{\sigma_{\perp}^2 + 3 \cdot \tau_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$181.19 \text{ MPa} < 435.56 \text{ MPa}$	0.416	✓
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The point where the stresses are checked	$z_i = 28.00 \text{ mm}$
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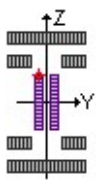
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Elastic modulus of welds	
$W_w = 104.97 \text{ cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-76.29 \text{ kN}}{18.16 \text{ cm}^2} = -42.01 \text{ MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{W_w} = \frac{-5.06 \text{ kNm} \cdot 28.00 \text{ mm}}{293.91 \text{ cm}^4} = -48.22 \text{ MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -42.01 \text{ MPa} + (-48.22 \text{ MPa}) = -90.23 \text{ MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-90.23 \text{ MPa}}{\sqrt{2}} = -63.80 \text{ MPa}$	
Perpendicular tangent stress	
$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-90.23 \text{ MPa}}{\sqrt{2}} = -63.80 \text{ MPa}$	
Parallel tangent stress	
$\tau_{\parallel} = \frac{V_0}{A_{ww}} = \frac{6.55 \text{ kN}}{18.16 \text{ cm}^2} = 14.62 \text{ MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -63.80 \text{ MPa} < 352.80 \text{ MPa}$	0.181	✓
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$\sqrt{\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$130.10 \text{ MPa} < 435.56 \text{ MPa}$	0.299	✓
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The point where the stresses are checked	$z_i = -28.00 \text{ mm}$
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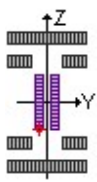
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Elastic modulus of welds	
$W_w = 104.97 \text{ cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-76.29 \text{ kN}}{18.16 \text{ cm}^2} = -42.01 \text{ MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{W_w} = \frac{-5.06 \text{ kNm} \cdot (-28.00 \text{ mm})}{293.91 \text{ cm}^4} = 48.22 \text{ MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -42.01 \text{ MPa} + 48.22 \text{ MPa} = 6.22 \text{ MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{6.22 \text{ MPa}}{\sqrt{2}} = 4.39 \text{ MPa}$	
Perpendicular tangent stress	
$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{6.22 \text{ MPa}}{\sqrt{2}} = 4.39 \text{ MPa}$	
Parallel tangent stress	
$\tau_{\parallel} = \frac{V_0}{A_{ww}} = \frac{6.55 \text{ kN}}{18.16 \text{ cm}^2} = 14.62 \text{ MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ 4.39 \text{ MPa} < 352.80 \text{ MPa}$	0.012	✓
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$\sqrt{\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$26.80 \text{ MPa} < 435.56 \text{ MPa}$	0.062	✓
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The point where the stresses are checked	$z_i = -50.00 \text{ mm}$
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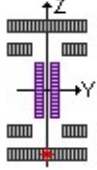
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Elastic modulus of welds	
$W_w = 58.78 \text{ cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-76.29 \text{ kN}}{18.16 \text{ cm}^2} = -42.01 \text{ MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{W_w} = \frac{-5.06 \text{ kNm} \cdot (-50.00 \text{ mm})}{293.91 \text{ cm}^4} = 86.11 \text{ MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -42.01 \text{ MPa} + 86.11 \text{ MPa} = 44.11 \text{ MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{44.11 \text{ MPa}}{\sqrt{2}} = 31.19 \text{ MPa}$	
Perpendicular tangent stress	
$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{44.11 \text{ MPa}}{\sqrt{2}} = 31.19 \text{ MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ 31.19 \text{ MPa} < 352.80 \text{ MPa}$	0.088	✓
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$\sqrt{\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$62.37 \text{ MPa} < 435.56 \text{ MPa}$	0.143	✓
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The rotational stiffness of a joint

The bolt elongation length

$$L_b = 8 \cdot d + t_p + t_{wa} + 0.5 \cdot m = 8 \cdot 10.00 \text{ mm} + 8.50 \text{ mm} + 2.00 \text{ mm} + 0.5 \cdot 9.50 \text{ mm} = 115.25 \text{ mm}$$

Bolt tension

$$k_{13} = \frac{E_c \cdot \sqrt{b_{eff} \cdot l_{eff}}}{1.275 \cdot E} = \frac{32800.00 \text{ MPa} \cdot \sqrt{49.35 \text{ mm} \cdot 141.35 \text{ mm}}}{1.275 \cdot 210000.00 \text{ MPa}} = 10.23 \text{ mm}$$

End plate in bending

$$k_{15} = \frac{0.425 \cdot l_{eff} \cdot t_p^3}{m_x^3} = \frac{0.425 \cdot 15.55 \text{ mm} \cdot (8.50 \text{ mm})^3}{(2.47 \text{ mm})^3} = 267.82 \text{ mm}$$

Bolt tension



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$$k_{16} = \frac{2 \cdot A_b}{L_b} = \frac{1.6 \cdot 0.58 \text{ cm}^2}{115.25 \text{ mm}} = 1.01 \text{ mm}$$

$$k_t = k_{15} + k_{16} = 267.82 \text{ mm} + 1.01 \text{ mm} = 268.83 \text{ mm}$$

$$k_c = k_{13} = 10.23 \text{ mm}$$

The initial rotational stiffness of a joint

$$S_{j,ini} = \frac{E \cdot z^2}{\frac{1}{k_c} + \frac{1}{k_t}} \cdot \frac{e}{e + e_k} = \frac{210000.00 \text{ MPa} \cdot (99.00 \text{ mm})^2}{\frac{1}{10.23 \text{ mm}} + \frac{1}{268.83 \text{ mm}}} \cdot \frac{66.35 \text{ mm}}{66.35 \text{ mm} + 51.37 \text{ mm}} = 11434.27 \frac{\text{kNm}}{\text{rad}}$$

$$S_{j,ini} = \frac{E \cdot z^2}{\frac{1}{k_c} + \frac{1}{k_t}} \cdot \frac{e}{e + e_k} = \frac{210000.00 \text{ MPa} \cdot (99.00 \text{ mm})^2}{\frac{1}{10.23 \text{ mm}} + \frac{1}{268.83 \text{ mm}}} \cdot \frac{66.35 \text{ mm}}{66.35 \text{ mm} + 51.37 \text{ mm}} = 11434.27 \frac{\text{kNm}}{\text{rad}}$$

The rotational stiffness of a pinned joint

$$S_{j,pin} = \frac{0.5 \cdot E \cdot I_{yc}}{L_c} = \frac{0.5 \cdot 210000.00 \text{ MPa} \cdot 349.24 \text{ cm}^4}{2320.00 \text{ mm}} = 158.06 \frac{\text{kNm}}{\text{rad}}$$

The rotational stiffness of a rigid joint

$$S_{j,rig} = \frac{30 \cdot E \cdot I_{yc}}{L_c} = \frac{30 \cdot 210000.00 \text{ MPa} \cdot 349.24 \text{ cm}^4}{2320.00 \text{ mm}} = 9483.58 \frac{\text{kNm}}{\text{rad}}$$

Stiffness ratio

$$\mu = \min(1.0; \frac{|M_{b1,Ed}|}{M_{Rd}}) = \min(1.0; \frac{-5.06 \text{ kNm}}{7.55 \text{ kNm}}) = 1.000$$

The rotational stiffness of a joint

$$S_j = \frac{S_{j,ini}}{\mu} = \frac{11434.27 \frac{\text{kNm}}{\text{rad}}}{1.000} = 11434.27 \frac{\text{kNm}}{\text{rad}}$$

Classification of joint

Rigid

The rotational stiffness of a joint

The bolt elongation length

$$L_b = 8 \cdot d + t_p + t_{wa} + 0.5 \cdot m = 8 \cdot 10.00 \text{ mm} + 8.50 \text{ mm} + 2.00 \text{ mm} + 0.5 \cdot 9.50 \text{ mm} = 115.25 \text{ mm}$$

Bolt tension

$$k_{13} = \frac{E_c \cdot \sqrt{b_{eff} \cdot l_{eff}}}{1.275 \cdot E} = \frac{32800.00 \text{ MPa} \cdot \sqrt{49.35 \text{ mm} \cdot 141.35 \text{ mm}}}{1.275 \cdot 210000.00 \text{ MPa}} = 10.23 \text{ mm}$$

End plate in bending



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$$k_{15} = \frac{0.425 \cdot l_{eff} \cdot t_p^3}{m_x^3} = \frac{0.425 \cdot 15.55mm \cdot (8.50mm)^3}{(2.47mm)^3} = 267.82mm$$

Bolt tension

$$k_{16} = \frac{2 \cdot A_b}{L_b} = \frac{1.6 \cdot 0.58cm^2}{115.25mm} = 1.01mm$$

$$k_t = k_{15} + k_{16} = 267.82mm + 1.01mm = 268.83mm$$

$$k_c = k_{13} = 10.23mm$$

The initial rotational stiffness of a joint

$$S_{j,ini} = \frac{E \cdot z^2}{\frac{1}{k_c} + \frac{1}{k_t}} \cdot \frac{e}{e + e_k} = \frac{210000.00MPa \cdot (95.00mm)^2}{\frac{1}{10.23mm} + \frac{1}{268.83mm}} \cdot \frac{3.13mm}{3.13mm + 0.00mm} = 9695.70 \frac{kNm}{rad}$$

$$S_{j,ini} = \frac{E \cdot z^2}{\frac{1}{k_c} + \frac{1}{k_t}} \cdot \frac{e}{e + e_k} = \frac{210000.00MPa \cdot (95.00mm)^2}{\frac{1}{10.23mm} + \frac{1}{268.83mm}} \cdot \frac{3.13mm}{3.13mm + 0.00mm} = 9695.70 \frac{kNm}{rad}$$

The rotational stiffness of a pinned joint

$$S_{j,pin} = \frac{0.5 \cdot E \cdot I_{yc}}{L_c} = \frac{0.5 \cdot 210000.00MPa \cdot 349.24cm^4}{2320.00mm} = 158.06 \frac{kNm}{rad}$$

The rotational stiffness of a rigid joint

$$S_{j,rig} = \frac{30 \cdot E \cdot I_{yc}}{L_c} = \frac{30 \cdot 210000.00MPa \cdot 349.24cm^4}{2320.00mm} = 9483.58 \frac{kNm}{rad}$$

Stiffness ratio

$$\mu = 1.000$$

The rotational stiffness of a joint

$$S_j = \frac{S_{j,ini}}{\mu} = \frac{9695.70 \frac{kNm}{rad}}{1.000} = 9695.70 \frac{kNm}{rad}$$

Classification of joint

Rigid



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STEEL MEMBER DESIGN

 Design member **3**

 Nodes: **16-19**

 Code: **Eurocode [LV]**

 Material: **S 355**

 Cross-section: **HEA 100**

 Load case: **Linear,(Auto) Critical**

 Coefficient for seismic forces: **1.0**

1. Axial force-Bending-Shear

EN 1993-1-1: 6.2.1, 6.2.8, 6.2.9

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 1.000 \cdot L = 1.000 \cdot 232 = 232$ cm

$$N_{Ed} = -76 \text{ kN} \quad V_{y,Ed} = -0.063 \text{ kN} \quad V_{z,Ed} = -6.5 \text{ kN} \quad M_{y,Ed} = -1013 \text{ kNcm} = -10.130 \text{ kNm} \quad M_{z,Ed} = -9.3 \text{ kNcm} = -0.093 \text{ kNm} \quad M_{x,Ed} = 0.0059 \text{ kNcm} = 0 \text{ kNm}$$

$$\eta_{NMV_{pl}} = \max(\eta_N; \eta_{M_{y,pl}}; \eta_{M_{z,pl}}; \eta_{V_z}; \eta_{V_y}) = \max(10.1; 34.4; 0.6; 4.2; 0) = 34.4 \% \quad \text{passed}$$

2. Axial Force-Bending-Flexural Buckling

EN 1993-1-1: 6.3.3, Annex B: Method 2

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 1.000 \cdot L = 1.000 \cdot 232 = 232$ cm

$$C_{my} = \max(0.2 + 0.8 \cdot \alpha_{my}, 0.4) = \max(0.2 + 0.8 \cdot 0.25, 0.4) = 0.4 \geq 0.4 \quad \text{Table B.3}$$

$$C_{mz} = \max(0.2 + 0.8 \cdot \alpha_{mz}, 0.4) = \max(0.2 + 0.8 \cdot 0.695, 0.4) = 0.756 \geq 0.4 \quad \text{Table B.3}$$

$$f_{yy} = \min(\lambda_y^* - 0.2; 0.8) = \min(0.75 - 0.2; 0.8) = 0.549$$

$$f_{zz} = \min(2 \cdot \lambda_z^* - 0.6; 1.4) = \min(2 \cdot 1.21 - 0.6; 1.4) = 1.4$$

$$k_{yy} = C_{my} \cdot \left(1 + f_{yy} \cdot \frac{|N_{Ed}|}{\frac{\chi_y \cdot N_{pl,Rd}}{\gamma_{M1}}} \right) = 0.4 \cdot \left(1 + 0.549 \cdot \frac{|(-76)|}{\frac{0.76 \cdot 754}{1}} \right) = 0.429$$

$$k_{zy} = 0.6 \cdot k_{yy} = 0.6 \cdot 0.429 = 0.258 \quad \text{Table Annex B.1}$$

$$k_{yz} = 0.6 \cdot k_{zz} = 0.6 \cdot 1.004 = 0.602$$

$$k_{zz} = C_{mz} \cdot \left(1 + f_{zz} \cdot \frac{|N_{Ed}|}{\frac{\chi_z \cdot N_{pl,Rd}}{\gamma_{M1}}} \right) = 0.756 \cdot \left(1 + 1.4 \cdot \frac{|(-76)|}{\frac{0.43 \cdot 754}{1}} \right) = 1.004 \quad \text{Table Annex B.1}$$

$$\chi_y = \min\left(\frac{1}{\phi_y + \sqrt{\phi_y^2 - \lambda_y^{*2}}}; 1\right) = 0.76 \quad (6.49)$$

$$\chi_z = \min\left(\frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_z^{*2}}}; 1\right) = 0.43 \quad (6.49)$$

$$\eta_{NMBuckl_1} = \frac{|N_{Ed}|}{\frac{\chi_y \cdot A \cdot f_y}{\gamma_{M1}}} + k_{yy} \cdot \frac{|M_{y,Ed}|}{\frac{W_{ply} \cdot f_y}{\gamma_{M1}}} + k_{yz} \cdot \frac{|M_{z,Ed}|}{\frac{W_{pl,z} \cdot f_y}{\gamma_{M1}}} = \frac{|(-76)|}{\frac{0.76 \cdot 21 \cdot 36}{1}} + 0.429 \cdot \frac{|(-1013)|}{\frac{83 \cdot 36}{1}} + 0.602 \cdot \frac{|(-9.3)|}{\frac{41 \cdot 36}{1}} = 28.5 \% \quad (6.61)$$



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$$\eta_{NMBuckl_2} = \frac{|N_{Ed}|}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}} + k_{zy} \cdot \frac{|M_{y,Ed}|}{\frac{W_{pl,y} \cdot f_y}{\gamma_{M1}}} + k_{zz} \cdot \frac{|M_{z,Ed}|}{\frac{W_{pl,z} \cdot f_y}{\gamma_{M1}}} = \frac{|(-76)|}{\frac{0.43 \cdot 21 \cdot 36}{1}} + 0.258 \cdot \frac{|(-1013)|}{\frac{83 \cdot 36}{1}} + 1.004 \cdot \frac{|(-9.3)|}{\frac{41 \cdot 36}{1}} = 32.9 \% \quad (6.62)$$

$$\eta_{NMBuckl} = 32.9 \% \quad \text{passed}$$

3. Axial force-Bending-Lateral torsional buckling

EN 1993-1-1: 6.3.3, Annex B: Method 2

Disabled

4. Cross-section resistance to shear (y):

EN 1993-1-1: 6.2.6, 6.2.7

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 0.000 \cdot L = 0.000 \cdot 232 = 0 \text{ cm}$

$$A_{V,y} = 2 \cdot b \cdot t_f = 16 \text{ cm}^2$$

$$V_{pl,Rd,y} = \frac{A_{V,y} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{16 \cdot 36}{\sqrt{3} \cdot 1} = 328 \text{ kN} \quad (6.18)$$

$$M_{x,Ed} = 0.0059 \text{ kNcm}$$

$$V_{pl,T,Rd,y} = \left(\sqrt{1 - \frac{\tau_{T,xy,Ed}}{1.25 \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}}}} \right) \cdot V_{pl,Rd,y} = \left(\sqrt{1 - \frac{0.00089}{1.25 \cdot \frac{36}{\sqrt{3} \cdot 1}}} \right) \cdot 328 = 328 \text{ kN} \quad (6.26)$$

$$\eta_{V_y} = \frac{|V_{y,Ed}|}{V_{pl,T,Rd,y}} = \frac{|(-0.063)|}{328} = 0 \% \quad (6.17) \quad \text{passed}$$

5. Shear web buckling resistance:

EN 1993-1-5: 5.1, 5.2, 5.3, 5.5, Annex A: A.3

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 0.000 \cdot L = 0.000 \cdot 232 = 0 \text{ cm}$

$$a_{max} = 2.32$$

$$\eta_w = 1.2 \quad 5.2 (2) \text{ Note 2}$$

$$\varepsilon = 0.81$$

$$h_w = h - 2 \cdot t_f = 9.6 - 2 \cdot 0.8 = 8 \text{ cm}$$

$$\frac{h_w}{t_w} \leq \frac{72 \cdot \varepsilon}{\eta_w} \rightarrow V_{b,Rd} = V_{pl,Rd,z} = 155 \text{ kN} \quad (5.1 (2))$$

$$\eta_{V_w} = \frac{|V_{z,Ed}|}{V_{b,Rd}} = \frac{|(-6.5)|}{155} = 4.2 \% \quad (5.10) \quad \text{passed}$$

6. Web shear-Bending-Axial force

EN 1993-1-1: 6.2.9; EN 1993-1-5: 7.1

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 1.000 \cdot L = 1.000 \cdot 232 = 232 \text{ cm}$

$$M_{f,Rd} = b \cdot t_f \cdot f_y \cdot (h - t_f) = 10 \cdot 0.8 \cdot 36 \cdot (9.6 - 0.8) = 2499 \text{ kNcm} = 24.992 \text{ kNm}$$



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$$\rho_{Nf} = \frac{1 - N_{Ed}}{\left(\frac{(b \cdot t_f + b_2 \cdot t_{f,2}) \cdot f_y}{\gamma_{M0}} \right)} = \frac{1 - (-76)}{\left(\frac{(10 \cdot 0.8 + 10 \cdot 0.8) \cdot 36}{1} \right)} = 0.8665$$

$$|M_{y,Ed}| \leq M_{f,Rd} \cdot \rho_{Nf} \rightarrow \eta_{V_{wMN}} = \frac{|M_{y,Ed}|}{M_{pl,Rd,y}} = \frac{|(-1013)|}{2947} = 34.4 \% \quad (7.1) \quad \text{passed}$$

7. SLS (Serviceability Limit State)

EN 1993-1-1: 7., EN 1990: 3.4, A1.4.

Critical combination: **[Gk] {Qk}**Section class: **1** (Plastic design)Critical section: $x = 0.700 \cdot L = 0.700 \cdot 232 = 162 \text{ cm}$

$$w_x = |w_{x,i} - w_{x,0}| = |(-0.065) - (-4.2356 \cdot 10^{-10})| = 0.065 \text{ cm}$$

$$w_{x,Limit} = \frac{H_{SLS}}{300.0} = \frac{232}{300.0} = 0.77 \text{ cm}$$

$$\eta_{w_x} = \frac{w_x}{w_{x,Limit}} = \frac{0.065}{0.77} = 8.4 \%$$

$$w_y = |w_{y,i} - w_{y,0}| = |(-0.18) - (-4.4115 \cdot 10^{-8})| = 0.18 \text{ cm}$$

$$w_{y,Limit} = \frac{H_{SLS}}{300.0} = \frac{232}{300.0} = 0.77 \text{ cm}$$

$$\eta_{w_y} = \frac{w_y}{w_{y,Limit}} = \frac{0.18}{0.77} = 23.8 \%$$

$$\eta_{SLS} = \max(\eta_{w_x}; \eta_{w_y}) = \max(8.4; 23.8) = 23.8 \% \quad \text{passed}$$

Partial results

8. Cross-section resistance to axial force:

EN 1993-1-1: 6.2.4

Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**Section class: **1** (Plastic design)Critical section: $x = 0.000 \cdot L = 0.000 \cdot 232 = 0 \text{ cm}$

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{21 \cdot 36}{1} = 754 \text{ kN} \quad (6.10)$$

$$\eta_N = \frac{|N_{Ed}|}{N_{pl,Rd}} = \frac{|(-76)|}{754} = 10.1 \% \quad (6.9) \quad \text{passed}$$

9. Cross-section resistance to bending (yy):

EN 1993-1-1: 6.2.5

Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**Section class: **1** (Plastic design)Critical section: $x = 1.000 \cdot L = 1.000 \cdot 232 = 232 \text{ cm}$

$$M_{pl,Rd,y} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{83 \cdot 36}{1} = 2947 \text{ kNcm} = 29.471 \text{ kNm} \quad (6.13)$$



$$\eta_{M_{y,pl}} = \frac{|M_{y,Ed}|}{M_{pl,Rd,y}} = \frac{|(-1013)|}{2947} = 34.4 \% \quad (6.12) \quad \text{passed}$$

10. Cross-section resistance to bending (zz):

EN 1993-1-1: 6.2.5

Critical combination: $[1.35 \cdot 0.85 \cdot Gk] \{1.5 \cdot Qk\}$

Section class: **1** (Plastic design)

Critical section: $x = 0.000 \cdot L = 0.000 \cdot 232 = 0$ cm

$$M_{pl,Rd,z} = \frac{W_{pl,z} \cdot f_y}{\gamma_{M0}} = \frac{41 \cdot 36}{1} = 1461 \text{ kNcm} = 14.605 \text{ kNm} \quad (6.13)$$

$$\eta_{M_{z,pl}} = \frac{|M_{z,Ed}|}{M_{pl,Rd,z}} = \frac{|(-24)|}{1461} = 1.6 \% \quad (6.12) \quad \text{passed}$$

11. Cross-section resistance to shear (z):

EN 1993-1-1: 6.2.6, 6.2.7

Critical combination: $[1.35 \cdot 0.85 \cdot Gk] \{1.5 \cdot Qk\}$

Section class: **1** (Plastic design)

Critical section: $x = 0.000 \cdot L = 0.000 \cdot 232 = 0$ cm

$$A_{V,z} = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f = 7.6 \text{ cm}^2$$

$$V_{pl,Rd,z} = \frac{A_{V,z} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{7.6 \cdot 36}{\sqrt{3} \cdot 1} = 155 \text{ kN} \quad (6.18)$$

$$M_{x,Ed} = 0.0059 \text{ kNcm}$$

$$V_{pl,T,Rd,z} = \left(\sqrt{1 - \frac{\tau_{T,xz,Ed}}{1.25 \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}}}} \right) \cdot V_{pl,Rd,z} = \left(\sqrt{1 - \frac{0.00056}{1.25 \cdot \frac{36}{\sqrt{3} \cdot 1}}} \right) \cdot 155 = 155 \text{ kN} \quad (6.26)$$

$$\eta_{V_z} = \frac{|V_{z,Ed}|}{V_{pl,T,Rd,z}} = \frac{|(-6.5)|}{155} = 4.2 \% \quad (6.17) \quad \text{passed}$$

12. Bending-shear interaction check

EN 1993-1-1: 6.2.1, 6.2.8, 6.2.9

Critical combination for N-M-V strength interaction: $[1.35 \cdot 0.85 \cdot Gk] \{1.5 \cdot Qk\}$

Section class: **1** (Plastic design)

Critical section: $x = 1.000 \cdot L = 1.000 \cdot 232 = 232$ cm

$$V_{z,Ed} = -6.5 \text{ kN} \leq V_{pl,Rd,z}/2 = 77 \text{ kN} \rightarrow \text{The effect of shear force on moment resistance is negligible.} \quad 6.2.8 (2)$$

$$V_{y,Ed} = -0.063 \text{ kN} \leq V_{pl,Rd,y}/2 = 164 \text{ kN} \rightarrow \text{The effect of shear force on moment resistance is negligible.} \quad 6.2.8 (2)$$

13. Bending-axial force interaction check

EN 1993-1-1: 6.2.1, 6.2.8, 6.2.9

Critical combination for N-M-V strength interaction: $[1.35 \cdot 0.85 \cdot Gk] \{1.5 \cdot Qk\}$

Section class: **1** (Plastic design)

Critical section: $x = 1.000 \cdot L = 1.000 \cdot 232 = 232$ cm

$$n = \frac{|N_{Ed}|}{N_{pl,Rd}} = \frac{76}{754} = 0.10 = 10.06\% \leq 25\%$$



$$|N_{Ed}| = 76 \text{ kN} \geq N_{lim,y} = \frac{h_w \cdot t_w \cdot f_y}{2 \cdot \gamma_{M0}} = \frac{8 \cdot 0.5 \cdot 36}{2 \cdot 1} = 71 \text{ kN}$$

$$|N_{Ed}| = 76 \text{ kN} \leq N_{lim,z} = \frac{h_w \cdot t_w \cdot f_y}{\gamma_{M0}} = \frac{8 \cdot 0.5 \cdot 36}{1} = 142 \text{ kN}$$

$$a_w = \min \left(\frac{A - 2 \cdot b \cdot t_f}{A}; 0.5 \right) = \min \left(\frac{21 - 2 \cdot 10 \cdot 0.8}{21}; 0.5 \right) = 0.25$$

$$\rho_{N_y} = \max \left(\frac{1 - n}{1 - 0.5 \cdot a_w}; 0.01 \right) = \max \left(\frac{1 - 0.10}{1 - 0.5 \cdot 0.25}; 0.01 \right) = 1.026$$

$$M_{N,y,Rd} = \min (M_{y,V,Rd} \cdot \rho_{N_y}; M_{y,V,Rd}) = \min (2947 \cdot 1.026; 2947) = 2947 \text{ kNcm} = 29.471 \text{ kNm}$$

$$M_{N,z,Rd} = M_{z,V,Rd} = 1461 \text{ kNcm} = 14.605 \text{ kNm}$$

$$\eta_{MN,1} = \frac{M_{y,Ed}}{M_{N,y,Rd}} = \frac{(-1013)}{2947} = 34.4 \%$$

$$\eta_{MN,2} = \frac{M_{z,Ed}}{M_{N,z,Rd}} = \frac{(-9.3)}{1461} = 0.6 \%$$

$$\alpha_{MN} = 2$$

$$\beta_{MN} = \max (5 \cdot n; 1) = \max (5 \cdot 0.10; 1) = 1$$

$$\eta_{MN,3} = \left(\frac{M_{y,Ed}}{M_{N,y,Rd}} \right)^{\alpha_{MN}} + \left(\frac{M_{z,Ed}}{M_{N,z,Rd}} \right)^{\beta_{MN}} = \left(\frac{(-1013)}{2947} \right)^2 + \left(\frac{(-9.3)}{1461} \right)^1 = 12.5 \% \quad (6.41)$$

$$\eta_{MN} = \max (\eta_{MN,1}; \eta_{MN,2}; \eta_{MN,3}; \eta_N) = \max (34.4; 0.6; 12.5; 10.1) = 34.4 \% \quad \text{passed}$$

14. Buckling resistance:

EN 1993-1-1: 6.3.1

Critical combination for N-M-Buckling interaction: $[1.35 \cdot 0.85 \cdot G_k] \{1.5 \cdot Q_k\}$

Section class: **1** (Plastic design)

Critical section: $x = 1.000 \cdot L = 1.000 \cdot 232 = 232 \text{ cm}$

$$K_y = 1$$

$$K_z = 1$$

$$L_{cr,y} = K_y \cdot L = 1 \cdot 232 = 232 \text{ cm}$$

$$L_{cr,z} = K_z \cdot L = 1 \cdot 232 = 232 \text{ cm}$$

Buckling curve about the y axis: *b* Table 6.2

$$\rightarrow \alpha_y = 0.34 \quad \text{Table 6.1}$$

Buckling curve about the z axis: *c* Table 6.2

$$\rightarrow \alpha_z = 0.49 \quad \text{Table 6.1}$$

$$\lambda_y^* = \sqrt{\frac{A \cdot f_y}{N_{cr,y}}} = \sqrt{\frac{21 \cdot 36}{1345}} = 0.75 \quad (6.50)$$

$$\lambda_z^* = \sqrt{\frac{A \cdot f_y}{N_{cr,z}}} = \sqrt{\frac{21 \cdot 36}{515}} = 1.21 \quad (6.50)$$

$$\phi_y = \frac{1 + \alpha_y \cdot (\lambda_y^* - 0.2) + \lambda_y^{*2}}{2} = \frac{1 + 0.34 \cdot (0.75 - 0.2) + 0.75^2}{2} = 0.8736$$



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$$\phi_z = \frac{1 + \alpha_z \cdot (\lambda_z^* - 0.2) + \lambda_z^{*2}}{2} = \frac{1 + 0.49 \cdot (1.21 - 0.2) + 1.21^2}{2} = 1.4789$$

$$\chi_y = \min \left(\frac{1}{\phi_y + \sqrt{\phi_y^2 - \lambda_y^{*2}}}; 1 \right) = \min \left(\frac{1}{0.8736 + \sqrt{0.8736^2 - 0.75^2}}; 1 \right) = 0.76 \quad (6.49)$$

$$\chi_z = \min \left(\frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_z^{*2}}}; 1 \right) = \min \left(\frac{1}{1.4789 + \sqrt{1.4789^2 - 1.21^2}}; 1 \right) = 0.43 \quad (6.49)$$

$$\chi = \min(\chi_y; \chi_z) = \min(0.76; 0.43) = 0.43 \leq 1.0$$

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.43 \cdot 21 \cdot 36}{1} = 324 \text{ kN} \quad (6.47)$$

$$\eta_{N_b} = \frac{|N_{Ed}|}{N_{b,Rd}} = \frac{|(-76)|}{324} = 23.4 \% \quad (6.46) \quad \text{passed}$$

15. Lateral-torsional buckling resistance:

EN 1993-1-1: 6.3.2

Disabled



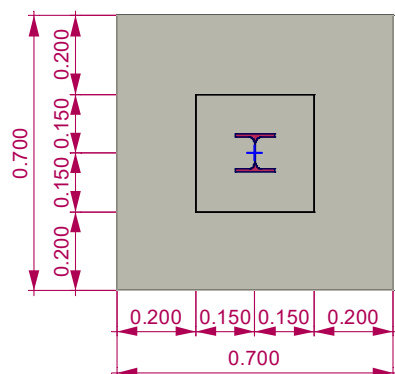
Pad footing design

Code: Eurocode [LV]

Support : NSup 1

1. Footing

Geometry:

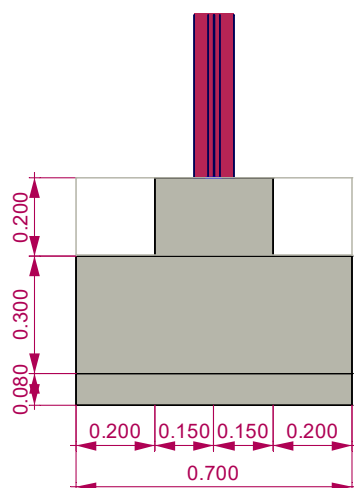


Materials

Concrete: C30/37 ($f_{ck} = 30$ MPa)density (concrete): $\rho_C = 2200$ kg/m³density (reinforced concrete): $\rho_{RC} = 2500$ kg/m³

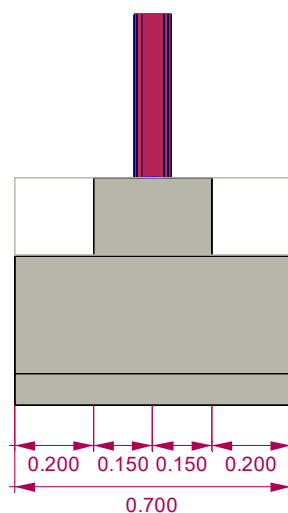
Rebar steel:

Longitudinal rebars: B500B

 $f_{yk} = 500$ MPa

0.000

-0.580

Embedment depth: $D = 0.58$ m

Characteristic value of weight density of materials:

Concrete: $\gamma_{C,k} = \rho_C \cdot g \cdot 10^{-3} = 2200 \cdot 9.810 \cdot 10^{-3} = 21.582$ kN/m³Reinforced concrete: $\gamma_{RC,k} = \rho_{RC} \cdot g \cdot 10^{-3} = 2500 \cdot 9.810 \cdot 10^{-3} = 24.525$ kN/m³

1.1. Footing



Width of pad footing: $b = b_x = 0.7 \text{ m}$

Length of pad footing: $L = b_y = 0.7 \text{ m}$

Width of the step:

$$b_{Tx} = b_{Ty} = 0.3 \text{ m}$$

Footing plate thickness: $h_p = 0.3 \text{ m}$

Step height: $h_s = 0.2 \text{ m}$

Height of footing: $h = h_p + h_s = 0.3 + 0.2 = 0.5 \text{ m}$

Area of the footing: $A = b^2 = 0.7^2 = 0.49 \text{ m}^2$

Volume of the footing:

$$V_f = A \cdot h_p + b_{Tx} \cdot b_{Ty} \cdot h_s = 0.49 \cdot 0.3 + 0.3 \cdot 0.3 \cdot 0.2 = 0.165 \text{ m}^3$$

Characteristic value of the weight of the footing:

$$G_{fk} = V_f \cdot \gamma_{RC,k} = 0.165 \cdot 24.525 = 4.0466 \text{ kN (↓)}$$

1.2. Blind concrete

Thickness of the blind concrete: $h_b = 0.08 \text{ m}$

Volume of the blind concrete:

$$V_b = A \cdot h_b = 0.49 \cdot 0.08 = 0.0392 \text{ m}^3$$

Characteristic value of the weight of blind concrete:

$$G_{bk} = V_b \cdot \gamma_{C,k} = 0.0392 \cdot 21.582 = 0.84601 \text{ kN (↓)}$$

1.3. Backfill

Material: Loose, dry sand (CSL)

Weight density of the backfill: $\gamma_{bf,k} = 16.677 \text{ kN/m}^3$

Volume of the part of the step below ground level:

$$V_{sp} = b_{Tx} \cdot b_{Ty} \cdot (D - h_b - h_p) = 0.3 \cdot 0.3 \cdot (0.58 - 0.08 - 0.3) = 0.018 \text{ m}^3$$

Volume of the part of the footing below ground level:

$$V_{fp} = A \cdot h_p + b_{Tx} \cdot b_{Ty} \cdot (D - h_b - h_p) = 0.49 \cdot 0.3 + 0.3 \cdot 0.3 \cdot (0.58 - 0.08 - 0.3) = 0.165 \text{ m}^3$$

Volume of the footing trench:

$$V_{ft} = A \cdot D = 0.49 \cdot 0.58 = 0.2842 \text{ m}^3$$

Volume of the backfill:

$$V_{bf} = V_{ft} - (V_{fp} + V_b) = 0.2842 - (0.165 + 0.0392) = 0.08 \text{ m}^3$$

Characteristic value of the weight of backfill:

$$G_{bfk} = V_{bf} \cdot \gamma_{bf,k} = 0.08 \cdot 16.677 = 1.3342 \text{ kN (↓)}$$

1.4. Footing (permanent action)

Characteristic value of the weight of the footing:

$$G_k = G_{fk} + G_{bk} + G_{bfk} = 4.0466 + 0.84601 + 1.3342 = 6.2268 \text{ kN (↓)}$$

Characteristic value of the weight of the footing: (without blind concrete)

$$G_k^* = G_{fk} + G_{bfk} = 4.0466 + 1.3342 = 5.3808 \text{ kN (↓)}$$

²without blind concrete

2. Soil parameters

2.1. Soil layers



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Name	Description	z_i [m]	h_i [m]	ρ_s [kg/m ³]	φ [°]	φ_{cv} [°]	c [kPa]	E_{oed} [kPa]	μ
Smilts		0	0.55	1900	18.00	18.00	—	2000	0
19pc		−0.55	1.3	2300	26.00	26.00	—	65000	0

where:

 z_i : Top level

 h_i : Thickness

 ρ_s : Density

 φ : Angle of shear resistance

 φ_{cv} : Critical state angle of shearing resistance

 c : Cohesion

 E_{oed} : Oedometer modulus of the soil layer

 μ : Poisson's ratio

2.2. Overburden pressure

Characteristic effective overburden pressure at the level of the foundation base:

$$q'_k = h_{s,1} \cdot \gamma_{s,1} + h_{s,2} \cdot \gamma_{s,2} = 0.55 \cdot 18.639 + 0.03 \cdot 22.563 = 10.928 \text{ kPa}$$

3. Bearing resistance calculation

Design Approach 2: {A1 "+" M1 "+" R2}

LVS EN 1997-1 Annex A

Partial factors			
A1	Permanent, unfavourable actions	$\gamma_{G,unfav}$	1.35
	Permanent, favourable actions	$\gamma_{G,fav}$	1
	Variable, unfavourable actions	$\gamma_{Q,unfav}$	1.5
	Variable, favourable actions	$\gamma_{Q,fav}$	0
M1	Angle of effective shear resistance	$\gamma_{\varphi'}$	1
	Effective cohesion	$\gamma_{c'}$	1
	Undrained shear strength	γ_{cu}	1
	Unconfined strength	γ_{qu}	1
	Weight density	γ_{γ}	1
R2	Bearing resistance	$\gamma_{R,v}$	1.4

3.1. Design value of parameters of the soil below the foundation

$$\text{Angle of effective shear resistance: } \varphi'_d = \arctan \frac{\tan \varphi'_k}{\gamma_{\varphi'}} = \arctan \frac{\tan 26.00^\circ}{1} = 26.00^\circ$$

$$\text{Weight density: } \gamma'_d = \frac{\gamma'_k}{\gamma_{\gamma}} = \frac{22.563}{1} = 22.563 \text{ kN/m}^3$$

3.2. Design value of loads at the top of the footing - Nodal support internal forces

Load case: [1.35*0.85*Gk] {1.5*Qk} (A1(b))

$$F_x = 0.1257 \text{ kN} \quad F_y = 0 \text{ kN} \quad F_z = -49.689 \text{ kN} \quad M_x = 0 \text{ kNm} \quad M_y = -0.2559 \text{ kNm}$$

$$\text{Vertical load: } V = -F_z = -(-49.689) = 49.689 \text{ kN} (\downarrow)$$



3.3. Design value of loads at the base of footing

Design value of the weight of the footing: $G_d = G_k \cdot \gamma_{G,unfav} = 6.2268 \cdot 1.35 = 8.4062 \text{ kN} (\downarrow)$

$$H_{dx} = F_x = 0.1257 \text{ kN}$$

$$H_{dy} = F_y = 0 \text{ kN} \quad H_d = \sqrt{H_{dx}^2 + H_{dy}^2} = \sqrt{0.1257^2 + 0^2} = 0.1257 \text{ kN}$$

$$V_d = V + G_d = 49.689 + 8.4062 = 58.095 \text{ kN} (\downarrow)$$

Eccentricity of the vertical load (V_d) relative to the center of the footing:

$$e_x = \frac{M_y + F_x \cdot (h + h_b)}{V_d} = \frac{(-0.2559) + 0.1257 \cdot (0.5 + 0.08)}{58.095} = -0.00315 \text{ m}$$

$$e_y = 0 \text{ m}$$

Effective width of the footing:

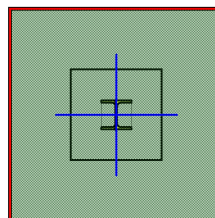
$$B' = \left(\frac{b_x}{2} - |e_x| \right) \cdot 2 = \left(\frac{0.7}{2} - |(-0.00315)| \right) \cdot 2 = 0.6937 \text{ m}$$

Effective length of the footing:

$$L' = \left(\frac{b_y}{2} - |e_y| \right) \cdot 2 = \left(\frac{0.7}{2} - |0| \right) \cdot 2 = 0.7 \text{ m}$$

Effective area of the footing:

$$A' = B' \cdot L' = 0.6937 \cdot 0.7 = 0.48559 \text{ m}^2$$



The design value of the effective overburden pressure at the level of the base:

$$q' = \frac{q'_k}{\gamma_\gamma} = \frac{10.928}{1} = 10.928 \text{ kPa}$$

3.4. Bearing resistance - Drained conditions

3.4.1. Dimensionless factors for the calculation [LVS EN 1997-1 Annex D D.4](#)

3.4.1.1. Bearing capacity factors

$$N_q = e^{\pi \cdot \tan \varphi'_d \cdot \tan^2 \left(45.00^\circ + \frac{\varphi'_d}{2} \right)} = e^{3.1416 \cdot \tan 26.00^\circ \cdot \tan^2 \left(45.00^\circ + \frac{26.00^\circ}{2} \right)} = 11.854$$

$$N_\gamma = 2 \cdot (N_q - 1) \cdot \tan \varphi'_d = 2 \cdot (11.854 - 1) \cdot \tan 26.00^\circ = 10.588$$

3.4.1.2. Factors for the inclination of the base

Inclination of the base: $\alpha_i = 0^\circ$

$$b_q = b_\gamma = 1$$

3.4.1.3. Shape factors of the footing base

$$s_q = 1 + \frac{B'}{L'} \cdot \sin \varphi'_d = 1 + \frac{0.6937}{0.7} \cdot \sin 26.00^\circ = 1.4344$$

$$s_\gamma = 1 - \frac{0.3 \cdot B'}{L'} = 1 - \frac{0.3 \cdot 0.6937}{0.7} = 0.7027$$

3.4.1.4. Factors for the inclination of the load

$$m_B = \frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}} = \frac{2 + \frac{0.6937}{0.7}}{1 + \frac{0.6937}{0.7}} = 1.5023 \quad m_L = \frac{2 + \frac{L'}{B'}}{1 + \frac{L'}{B'}} = \frac{2 + \frac{0.7}{0.6937}}{1 + \frac{0.7}{0.6937}} = 1.4977$$



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$$H_B = F_x = 0.1257 \text{ kN} \quad H_L = F_y = 0 \text{ kN}$$

$$\Theta = \arctan \frac{H_B}{H_L} = \arctan \frac{0.1257}{0} = 90.00^\circ$$

$$m = m_B = 1.5023$$

$$i_\gamma = \left(1 - \frac{H_d}{V_d + A' \cdot c'_d \cdot \cot \varphi'_d} \right)^{(m+1)} = \left(1 - \frac{0.1257}{58.095 + 0.48559 \cdot 0 \cdot \cot 26.00^\circ} \right)^{(1.5023+1)} = 0.99459$$

$$i_q = \left(1 - \frac{H_d}{V_d + A' \cdot c'_d \cdot \cot \varphi'_d} \right)^m = \left(1 - \frac{0.1257}{58.095 + 0.48559 \cdot 0 \cdot \cot 26.00^\circ} \right)^{1.5023} = 0.99675$$

3.5. Bearing resistance

$$R_d = \frac{A' \cdot (q' \cdot N_q \cdot b_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma'_d \cdot B' \cdot N_\gamma \cdot b_\gamma \cdot s_\gamma \cdot i_\gamma)}{\gamma_{R,v}} =$$

$$= \frac{0.48559 \cdot (10.928 \cdot 11.854 \cdot 1 \cdot 1.4344 \cdot 0.99675 + 0.5 \cdot 22.563 \cdot 0.6937 \cdot 10.588 \cdot 1 \cdot 0.7027 \cdot 0.99459)}{1.4} = 84.331 \text{ kN}$$

3.6. Bearing utilization

$$\Lambda_{R,v} = \frac{V_d}{R_d} = \frac{58.095}{84.331} = 0.68889 < \Lambda_{R,v,lim} = 1 \quad \text{passed}$$

4. Eccentricity check

Eccentricity limit factor: $\gamma_{ecc,lim} = 0.33$

F_x F_y [kN]	F_z [kN]	M_x M_y [kNm]	V_d [kN]	e_x e_y [m]	γ_{ecc}	✓ ✗	Load case
0 0	-1.79	0 -0.007	8.02	0 0	0.001	✓	[Gk]
0 0	-1.79	0 -0.007	8.02	0 0	0.001	✓	[Gk]
0.13 0	-49.69	0 -0.256	55.92	0 0	0.005	✓	[1.35*0.85*Gk] {1.5*Qk}
0.13 0	-49.69	0 -0.256	55.92	0 0	0.005	✓	[1.35*0.85*Gk] {1.5*Qk}
0 0	-1.79	0 -0.007	8.02	0 0	0.001	✓	[Gk]
0.11 0	-43.07	0 -0.222	49.3	0 0	0.005	✓	[Gk] {1.3*Qk}

where:

$$V_d = -F_z + G_k \cdot \gamma_{G,unfav}$$

$$\gamma_{ecc} = \sqrt{\left(\frac{e_x}{b_x} \right)^2 + \left(\frac{e_y}{b_y} \right)^2}$$

Eccentricity values take into account additional moment at the base of footing from horizontal

loads

 $\gamma_{G,unfav}$: Partial factor for footing self weight

The critical eccentricity

4.1. Design value of loads at the top of the footing - Nodal support internal forces



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Load case: $[1.35 \cdot 0.85 \cdot G_k] \{1.5 \cdot Q_k\}$ (A1(b))

$$F_x = 0.1257 \text{ kN} \quad F_y = 0 \text{ kN} \quad F_z = -49.689 \text{ kN} \quad M_x = 0 \text{ kNm} \quad M_y = -0.2559 \text{ kNm}$$

Vertical load: $V = -F_z = -(-49.689) = 49.689 \text{ kN} (\downarrow)$

4.2. Design value of loads at the base of footing

Design value of the weight of the footing: $G_d = G_k \cdot \gamma_{G, fav} = 6.2268 \cdot 1 = 6.2268 \text{ kN} (\downarrow)$

$$H_{dx} = F_x = 0.1257 \text{ kN}$$

$$H_{dy} = F_y = 0 \text{ kN}$$

$$V_d = V + G_d = 49.689 + 6.2268 = 55.916 \text{ kN} (\downarrow)$$

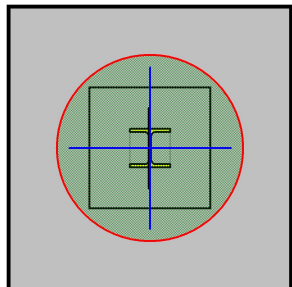
Eccentricity of the vertical load (V_d) relative to the center of the footing:

$$e_x = \frac{M_y + F_x \cdot (h + h_b)}{V_d} = \frac{(-0.2559) + 0.1257 \cdot (0.5 + 0.08)}{55.916} = -0.00327 \text{ m}$$

$$e_y = 0 \text{ m}$$

Ratio of eccentricity to the footing size:

$$\gamma_{ecc, max} = 0.0046714 < \gamma_{ecc, lim} = 0.33 \quad \text{passed}$$



5. Stability check

5.1. The axis of overturning

Ratio of distance between the axis of overturning and the footing edge to the footing size: $\gamma_{\omega} = 0.1$

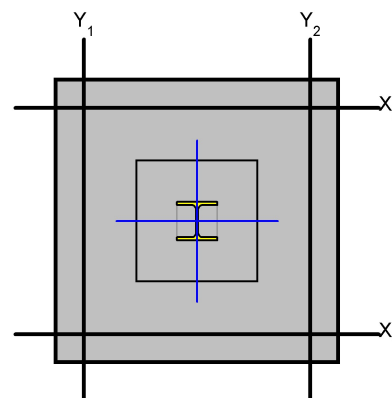
Distance between the axis of overturning and the center of the footing:

$$e_{EQU,y2} = b_y \cdot (0.5 - \gamma_{\omega}) = 0.7 \cdot (0.5 - 0.1) = 0.28 \text{ m}$$

$$e_{EQU,y1} = -e_{EQU,y2} = -0.28 \text{ m}$$

$$e_{EQU,x2} = b_x \cdot (0.5 - \gamma_{\omega}) = 0.7 \cdot (0.5 - 0.1) = 0.28 \text{ m}$$

$$e_{EQU,x1} = -e_{EQU,x2} = -0.28 \text{ m}$$



5.2. Stabilizing effect of the footing

Partial factor of permanent favourable (stabilizing) actions: $\gamma_{G,stab} = 0.9$

Stabilizing moment of the footing self-weight:

$$M_{G,stab,x1} = \gamma_{G,stab} \cdot G_k \cdot e_{EQU,y1} = 0.9 \cdot 6.2268 \cdot (-0.28) = -1.5692 \text{ kNm}$$

$$M_{G,stab,x2} = \gamma_{G,stab} \cdot G_k \cdot e_{EQU,y2} = 0.9 \cdot 6.2268 \cdot 0.28 = 1.5692 \text{ kNm}$$

$$M_{G,stab,y1} = \gamma_{G,stab} \cdot G_k \cdot (-e_{EQU,x1}) = 0.9 \cdot 6.2268 \cdot (-(-0.28)) = 1.5692 \text{ kNm}$$

$$M_{G,stab,y2} = \gamma_{G,stab} \cdot G_k \cdot (-e_{EQU,x2}) = 0.9 \cdot 6.2268 \cdot (-0.28) = -1.5692 \text{ kNm}$$

5.3. Checking rotation about the x_1 axis

5.3.1. Design value of loads at the top of the footing - Nodal support internal forces

Load case: [0.9*Gk] (EQU)

$$F_x = 0.0035012 \text{ kN} \quad F_y = 0 \text{ kN} \quad F_z = -1.6141 \text{ kN} \quad M_x = 0 \text{ kNm} \quad M_y = -0.006636 \text{ kNm}$$

Vertical load: $V = -F_z = -(-1.6141) = 1.6141 \text{ kN} (\downarrow)$

Stabilizing moment:

$$M_{x1,stab} = M_{G,stab,x1} + V \cdot e_{EQU,y1} = (-1.5692) + 1.6141 \cdot (-0.28) = -2.0211 \text{ kNm}$$

Destabilizing moment:

$$M_{x1,dst} = 0 \text{ kNm}$$

Utilization for stability:

$$\Lambda_{EQU,x1} = \left| \frac{M_{x1,dst}}{M_{x1,stab}} \right| = \left| \frac{0}{(-2.0211)} \right| = 0 < \Lambda_{EQU,lim} = 1.000 \quad \checkmark$$

5.4. Checking rotation about the x_2 axis

5.4.1. Design value of loads at the top of the footing - Nodal support internal forces

Load case: [0.9*Gk] (EQU)

$$F_x = 0.0035012 \text{ kN} \quad F_y = 0 \text{ kN} \quad F_z = -1.6141 \text{ kN} \quad M_x = 0 \text{ kNm} \quad M_y = -0.006636 \text{ kNm}$$

Vertical load: $V = -F_z = -(-1.6141) = 1.6141 \text{ kN} (\downarrow)$

Stabilizing moment:



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$$M_{x2,stab} = M_{G,stab,x2} + V \cdot e_{EQU,y2} = 1.5692 + 1.6141 \cdot 0.28 = 2.0211 \text{ kNm}$$

Destabilizing moment:

$$M_{x2,dst} = 0 \text{ kNm}$$

Utilization for stability:

$$\Lambda_{EQU,x2} = \left| \frac{M_{x2,dst}}{M_{x2,stab}} \right| = \left| \frac{0}{2.0211} \right| = 0 < \Lambda_{EQU,lim} = 1.000 \quad \checkmark$$

5.5. Checking rotation about the y_1 axis

5.5.1. Design value of loads at the top of the footing - Nodal support internal forces

Load case: [1.1*Gk] {1.5*Qk} (EQU)

$$F_x = 0.12551 \text{ kN} \quad F_y = 0 \text{ kN} \quad F_z = -49.604 \text{ kN} \quad M_x = 0 \text{ kNm} \quad M_y = -0.25555 \text{ kNm}$$

Vertical load: $V = -F_z = -(-49.604) = 49.604 \text{ kN} (\downarrow)$

Stabilizing moment:

$$M_{y1,stab} = M_{G,stab,y1} - V \cdot e_{EQU,x1} + F_x \cdot (h_b + h) = 1.5692 - 49.604 \cdot (-0.28) + 0.12551 \cdot (0.08 + 0.5) = 15.531 \text{ kNm}$$

Destabilizing moment:

$$M_{y1,dst} = M_y = -0.25555 \text{ kNm}$$

Utilization for stability:

$$\Lambda_{EQU,y1} = \left| \frac{M_{y1,dst}}{M_{y1,stab}} \right| = \left| \frac{(-0.25555)}{15.531} \right| = 0.016 < \Lambda_{EQU,lim} = 1.000 \quad \checkmark$$

5.6. Checking rotation about the y_2 axis

5.6.1. Design value of loads at the top of the footing - Nodal support internal forces

Load case: [1.1*Gk] {1.5*Qk} (EQU)

$$F_x = 0.12551 \text{ kN} \quad F_y = 0 \text{ kN} \quad F_z = -49.604 \text{ kN} \quad M_x = 0 \text{ kNm} \quad M_y = -0.25555 \text{ kNm}$$

Vertical load: $V = -F_z = -(-49.604) = 49.604 \text{ kN} (\downarrow)$

Stabilizing moment:

$$M_{y2,stab} = M_{G,stab,y2} + M_y - V \cdot e_{EQU,x2} = (-1.5692) + (-0.25555) - 49.604 \cdot 0.28 = -15.714 \text{ kNm}$$

Destabilizing moment:

$$M_{y2,dst} = F_x \cdot (h_b + h) = 0.12551 \cdot (0.08 + 0.5) = 0.072797 \text{ kNm}$$

Utilization for stability:

$$\Lambda_{EQU,y2} = \left| \frac{M_{y2,dst}}{M_{y2,stab}} \right| = \left| \frac{0.072797}{(-15.714)} \right| = 0.005 < \Lambda_{EQU,lim} = 1.000 \quad \checkmark$$

5.7. Maximum utilization

$$\Lambda_{EQU,max} = \Lambda_{EQU,y1} = 0.016 < \Lambda_{EQU,lim} = 1.000 \quad \text{passed}$$

6. Checking uplift

6.1. Design value of loads at the top of the footing - Nodal support internal forces

Load case: [0.9*Gk] (EQU)

$$F_x = 0.0035012 \text{ kN} \quad F_y = 0 \text{ kN} \quad F_z = -1.6141 \text{ kN} \quad M_x = 0 \text{ kNm} \quad M_y = -0.006636 \text{ kNm}$$



Vertical load: $V = -F_z = -(-1.6141) = 1.6141 \text{ kN} (\downarrow)$

$V = -F_z = -(-1.6141) = 1.6141 \text{ kN} (\downarrow) \geq 0$ Vertical action on the foundation does not have a destabilizing effect

Design value of the combination of destabilizing permanent and variable vertical actions:

$$V_{dst,d} = 0 \text{ kN}$$

Design value of the stabilizing permanent vertical actions:

$$G_{stb,d} = \gamma_{G,stb} \cdot (G_{fk} + G_{bfk}) = 0.9 \cdot (4.0466 + 1.3342) = 4.8427 \text{ kN} (\downarrow)$$

$$\Lambda_{UPL} = \frac{V_{dst,d}}{G_{stb,d}} = \frac{0}{4.8427} = 0 < \Lambda_{UPL,lim} = 1 \text{ passed}$$

7. Sliding calculation

7.1. Design value of loads

Design situation: Persistent and transient

Load case: $[1.35 \cdot 0.85 \cdot G_k] \{1.5 \cdot Q_k\}$ (A1(b))

Design value of loads at the top of the footing:

$$F_x = 0.1257 \text{ kN} \quad F_y = 0 \text{ kN}$$

$$F = \sqrt{F_x^2 + F_y^2} = \sqrt{0.1257^2 + 0^2} = 0.1257 \text{ kN}$$

$$V = -F_z = -(-49.689) = 49.689 \text{ kN} (\downarrow)$$

$$M_x = 0 \text{ kNm} \quad M_y = -0.2559 \text{ kNm}$$

$$M = \sqrt{M_x^2 + M_y^2} = \sqrt{0^2 + (-0.2559)^2} = 0.2559 \text{ kNm}$$

7.2. Sliding of the footing at the soil

Design Approach 2: {A1 "+" M1 "+" R2}

LVS EN 1997-1 Annex A

Partial factors			
A1	Permanent, unfavourable actions	$\gamma_{G,unfav}$	1.35
	Permanent, favourable actions	$\gamma_{G,fav}$	1
	Variable, unfavourable actions	$\gamma_{Q,unfav}$	1.5
	Variable, favourable actions	$\gamma_{Q,fav}$	0
M1	Angle of effective shear resistance	$\gamma_{\phi'}$	1
	Effective cohesion	$\gamma_{c'}$	1
	Undrained shear strength	γ_{cu}	1
	Unconfined strength	γ_{qu}	1
	Weight density	γ_γ	1
R2	Sliding resistance	$\gamma_{R,h}$	1.1

7.2.1. Design value of parameters of the soil below the foundation

Soil layer density: $\rho_s = 2300 \text{ kg/m}^3$

Weight density:

$$\gamma' = \frac{\rho_s \cdot g}{\gamma_\gamma} \cdot 10^{-3} = \frac{2300 \cdot 9.810}{1} \cdot 10^{-3} = 22.563 \text{ kN/m}^3$$

Critical state angle of shearing resistance: $\phi_{cv} = 26.00^\circ$

Design value of loads at the base of footing:

$$V_d = V + G_k \cdot \gamma_{G,fav} = 49.689 + 6.2268 \cdot 1 = 55.916 \text{ kN} (\downarrow)$$



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Design value of the horizontal force:

$$H_{dx} = F_x = 0.1257 \text{ kN}$$

$$H_{dy} = F_y = 0 \text{ kN}$$

$$H_d = \sqrt{F_x^2 + F_y^2} = \sqrt{0.1257^2 + 0^2} = 0.1257 \text{ kN}$$

7.2.1.1. Drained sliding resistance

Characteristic value of the structure-ground interface friction angle:

$$\delta_k = \varphi_{cv} = 26.00^\circ \quad (\text{cast-in-situ concrete foundation}) \quad \text{LVS EN 1997-1 6.5.3 (10)}$$

7.2.1.2. Shear resistance

$$R_d = \frac{V_{d'} \cdot \tan \delta_k}{\gamma_{R,h}} = \frac{55.916 \cdot \tan 26.00^\circ}{1.1} = 24.793 \text{ kN} \quad \text{LVS EN 1997-1 6.5.3 (8)P (6.3b)}$$

7.2.1.3. Sliding utilization

$$\Lambda_{R,h,s} = \left| \frac{H_d}{R_d} \right| = \left| \frac{0.1257}{24.793} \right| = 0.0050699 < \Lambda_{R,h,s,lim} = 1 \quad \text{passed}$$

7.3. Sliding of the footing on the blind concrete

Design Approach 2: {A1 "+" M1 "+" R2} LVS EN 1997-1 Annex A

Partial factors			
A1	Permanent, unfavourable actions	$\gamma_{G,unfav}$	1.35
	Permanent, favourable actions	$\gamma_{G,fav}$	1
	Variable, unfavourable actions	$\gamma_{Q,unfav}$	1.5
	Variable, favourable actions	$\gamma_{Q,fav}$	0
M1	Angle of effective shear resistance	$\gamma_{\phi'}$	1
	Effective cohesion	$\gamma_{c'}$	1
	Undrained shear strength	γ_{cu}	1
	Unconfined strength	γ_{qu}	1
	Weight density	γ_γ	1

 Coefficient of friction between the footing and the blind concrete: $\mu_{cc} = 0.7$

 Partial factor for friction between elements: $\gamma_\mu = 1$

Design value of loads at the top of the blind concrete:

$$V_d = V + G_k^* \cdot \gamma_{G,fav} = 49.689 + 5.3808 \cdot 1 = 55.07 \text{ kN} \quad (\downarrow)$$

Design value of the horizontal force:

$$H_{dx} = F_x = 0.1257 \text{ kN}$$

$$H_{dy} = F_y = 0 \text{ kN}$$

$$H_d = \sqrt{F_x^2 + F_y^2} = \sqrt{0.1257^2 + 0^2} = 0.1257 \text{ kN}$$

7.3.1. Sliding resistance

Friction resistance at the base of the footing:

$$R_{d,Hb} = \frac{V_d \cdot \mu_{cc}}{\gamma_\mu} = \frac{55.07 \cdot 0.7}{1} = 38.549 \text{ kN}$$



Sliding utilization:

$$\Lambda_{R,h,b} = \left| \frac{H_d}{R_{d,Hb}} \right| = \left| \frac{0.1257}{38.549} \right| = 0.003 < \Lambda_{R,h,b,lim} = 1.000 \quad \text{passed}$$

8. Structural investigation of the footing

8.1. Reinforcement design

Longitudinal rebars: $\varnothing 12 \text{ mm}$ ($A_{\varnothing} = 113 \text{ mm}^2$)

Rebar position:

$$u_{T,x} = 0.036 \text{ m}$$

$$u_{T,y} = 0.048 \text{ m}$$

$$u_{B,x} = 0.036 \text{ m}$$

$$u_{B,y} = 0.048 \text{ m}$$

8.2. Design of reinforcement for bending moment

Characteristic yield strength of rebar steel:

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

Factor, defining the effective height of the compression zone:

$$\lambda = 0.8 \quad \text{LVS EN 1992-1-1 (3.19.)}$$

Factor, defining the effective strength:

$$\eta = 1 \quad \text{LVS EN 1992-1-1 (3.21.)}$$

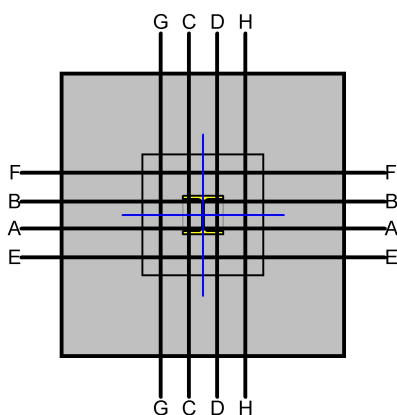
$$\varepsilon_{c1} = 0.0021619$$

The ultimate compressive strain in the concrete:

$$\varepsilon_{cu3} = 0.0035 \quad \text{LVS EN 1992-1-1 3.1.7. (2)}$$

The design value of the modulus of elasticity of reinforcing steel: $E_s = 200 \text{ GPa} = 2 \cdot 10^8 \text{ kPa}$

Moments in investigated cross-sections		
Cross-section	m_d [kNm/m]	Load case
A-A	5.1757	[1.35*0.85*Gk] {1.5*Qk}
B-B	5.1757	[1.35*0.85*Gk] {1.5*Qk}
C-C	5.13	[1.35*0.85*Gk] {1.5*Qk}
D-D	4.9269	[1.35*0.85*Gk] {1.5*Qk}
E-E	2.5595	[1.35*0.85*Gk] {1.5*Qk}
F-F	2.5595	[1.35*0.85*Gk] {1.5*Qk}
G-G	2.5595	[1.35*0.85*Gk] {1.5*Qk}
H-H	2.4166	[1.35*0.85*Gk] {1.5*Qk}



Design situation: Persistent and transient

Partial factor for concrete: $\gamma_c = 1.5$ LVS EN 1992-1-1 2.4.2.4. (1)



Partial factor for reinforcing steel: $\gamma_s = 1.15$ LVS EN 1992-1-1 2.4.2.4. (1)

The coefficient taking account of long term effects on the compressive strength and of unfavourable effects, resulting from the way the load is applied:

$$\alpha_{cc} = 1$$

Value of the concrete design compressive strength:

$$f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_c} = \frac{1 \cdot 30}{1.5} = 20 \text{ MPa} = 20000 \text{ kPa} \quad \text{LVS EN 1992-1-1 3.1.6. (1)P (3.15)}$$

Design value of the effective concrete compressive strength:

$$f_{cd,eff} = \eta \cdot f_{cd} = 1 \cdot 20000 = 20000 \text{ kPa} \quad \text{LVS EN 1992-1-1 3.1.7. (3)}$$

Design yield strength of rebar steel:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1.15} = 434.78 \approx 435 \text{ MPa} = 435000 \text{ kPa} \quad \text{LVS EN 1992-1-1 3.2.7. (2) Fig. 3.8}$$

$$\xi_0 = \frac{\varepsilon_{cu1}}{\varepsilon_{cu1} + \frac{f_{yd}}{E_s}} \cdot \lambda = \frac{0.0035}{0.0035 + \frac{435000}{2 \cdot 10^8}} \cdot 0.8 = 0.49339$$

8.2.1. Design of longitudinal reinforcement for M_y bending moment

8.2.1.1. Investigation at the edge of the supported element - cross-section C-C and D-D

Tension reinforcement in x direction

$$m_d = \text{Max}(m_{dC} ; m_{dD}) = \text{Max}(5.13 ; 4.9269) = 5.13 \text{ kNm/m}$$

$$d = h - u_{Bx} = 0.5 - 0.036 = 0.464 \text{ m}$$

$$x_{c0} = \xi_0 \cdot d = 0.49339 \cdot 0.464 = 0.22893 \text{ m}$$

$$x_c = d - \sqrt{d^2 - \frac{2 \cdot m_d}{f_{cd,eff}}} = 0.464 - \sqrt{0.464^2 - \frac{2 \cdot 5.13}{20000}} = 0.00055313 \text{ m} < x_{c0} = \xi_0 \cdot d = 0.49339 \cdot 0.464 = 0.22893 \text{ m}$$

Tension reinforcement area:

The minimum area of longitudinal tension reinforcement: LVS EN 1992-1-1 9.2.2.1. (1) (9.1N)

$$a_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot d = 0.26 \cdot \frac{2.8965}{500} \cdot 0.464 = 0.00069886 \text{ m}^2/\text{m} > 0.0013 \cdot d = 0.0013 \cdot 0.464 = 0.0006032 \text{ m}^2/\text{m} = 699$$

mm²/m

$$a_{s,1} = \frac{x_c \cdot f_{cd,eff}}{f_{yd}} = \frac{0.00055313 \cdot 20000}{435000} = 2.5431 \cdot 10^{-5} \text{ m}^2/\text{m} = 25 \text{ mm}^2/\text{m}$$

Minimum longitudinal tension reinforcement is not taken into account.

$$s_{max,slabs} = 2 \cdot h = 2 \cdot 0.5 = 1 \text{ m} > 0.25 \text{ m} \rightarrow s_{max,slabs} = 0.25 \text{ m} \quad \text{LVS EN 1992-1-1 9.3.1.1 (3)}$$

$$s = \frac{A_\emptyset}{a_{s,1}} = \frac{0.0001131}{2.5431 \cdot 10^{-5}} = 4.4472 \text{ m} > s_{max,slabs} = 0.25 \text{ m} \quad \text{!!} \rightarrow s = s_{max,slabs} = 0.25 \text{ m}$$

Longitudinal rebars: $\emptyset 12 \text{ mm}/250 \text{ mm}$

$$a_{s1,prov} = \frac{A_\emptyset}{s} = \frac{0.0001131}{0.25} = 0.00045239 \text{ m}^2/\text{m} = 452 \text{ mm}^2/\text{m} > a_{s,1} = 25 \text{ mm}^2/\text{m} \quad \checkmark$$

8.2.1.2. Investigation at the edge of the step - cross-section G and H

Tension reinforcement in x direction

$$m_d = \text{Max}(m_{dG} ; m_{dH}) = \text{Max}(2.5595 ; 2.4166) = 2.5595 \text{ kNm/m}$$

$$d = h - u_{Bx} = 0.3 - 0.036 = 0.264 \text{ m}$$



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$$x_{c0} = \xi_0 \cdot d = 0.49339 \cdot 0.264 = 0.13026 \text{ m}$$

$$x_c = d - \sqrt{d^2 - \frac{2 \cdot m_d}{f_{cd,eff}}} = 0.264 - \sqrt{0.264^2 - \frac{2 \cdot 2.5595}{20000}} = 0.0004852 \text{ m} < x_{c0} = \xi_0 \cdot d = 0.49339 \cdot 0.264 = 0.13026 \text{ m}$$

Tension reinforcement area:

The minimum area of longitudinal tension reinforcement: **LVS EN 1992-1-1 9.2.2.1. (1) (9.1N)**

$$a_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot d = 0.26 \cdot \frac{2.8965}{500} \cdot 0.264 = 0.00039763 \text{ m}^2/\text{m} > 0.0013 \cdot d = 0.0013 \cdot 0.264 = 0.0003432 \text{ m}^2/\text{m} = 398$$

mm²/m

$$a_{s,1} = \frac{x_c \cdot f_{cd,eff}}{f_{yd}} = \frac{0.0004852 \cdot 20000}{435000} = 2.2308 \cdot 10^{-5} \text{ m}^2/\text{m} = 22 \text{ mm}^2/\text{m} < a_{s,min} = 398 \text{ mm}^2/\text{m} \rightarrow a_{s,1} = a_{s,min} = 398$$

mm²/m

$$s_{max,slabs} = 2 \cdot h = 2 \cdot 0.3 = 0.6 \text{ m} > 0.25 \text{ m} \rightarrow s_{max,slabs} = 0.25 \text{ m} \quad \text{LVS EN 1992-1-1 9.3.1.1 (3)}$$

$$s = \frac{A_{\varnothing}}{a_{s,1}} = \frac{0.0001131}{0.00039763} = 0.28443 \text{ m} > s_{max,slabs} = 0.25 \text{ m} \quad \text{!!} \rightarrow s = s_{max,slabs} = 0.25 \text{ m}$$

Longitudinal rebars: $\varnothing 12 \text{ mm}/250 \text{ mm}$

$$a_{s1,prov} = \frac{A_{\varnothing}}{s} = \frac{0.0001131}{0.25} = 452 \text{ mm}^2/\text{m} > a_{s,1} = 398 \text{ mm}^2/\text{m} \quad \checkmark$$

8.2.2. Design of longitudinal reinforcement for M_x bending moment

8.2.2.1. Investigation at the edge of the supported element - cross-section A-A and B-B

Tension reinforcement in y direction

$$m_d = \text{Max}(m_{dA} ; m_{dB}) = \text{Max}(5.1757 ; 5.1757) = 5.1757 \text{ kNm/m}$$

$$d = h - u_{By} = 0.5 - 0.048 = 0.452 \text{ m}$$

$$x_{c0} = \xi_0 \cdot d = 0.49339 \cdot 0.452 = 0.22301 \text{ m}$$

$$x_c = d - \sqrt{d^2 - \frac{2 \cdot m_d}{f_{cd,eff}}} = 0.452 - \sqrt{0.452^2 - \frac{2 \cdot 5.1757}{20000}} = 0.0005729 \text{ m} < x_{c0} = \xi_0 \cdot d = 0.49339 \cdot 0.452 = 0.22301 \text{ m}$$

Tension reinforcement area:

The minimum area of longitudinal tension reinforcement: **LVS EN 1992-1-1 9.2.2.1. (1) (9.1N)**

$$a_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot d = 0.26 \cdot \frac{2.8965}{500} \cdot 0.452 = 0.00068079 \text{ m}^2/\text{m} > 0.0013 \cdot d = 0.0013 \cdot 0.452 = 0.0005876 \text{ m}^2/\text{m} = 681$$

mm²/m

$$a_{s,1} = \frac{x_c \cdot f_{cd,eff}}{f_{yd}} = \frac{0.0005729 \cdot 20000}{435000} = 2.634 \cdot 10^{-5} \text{ m}^2/\text{m} = 26 \text{ mm}^2/\text{m}$$

Minimum longitudinal tension reinforcement is not taken into account.

$$s_{max,slabs} = 2 \cdot h = 2 \cdot 0.5 = 1 \text{ m} > 0.25 \text{ m} \rightarrow s_{max,slabs} = 0.25 \text{ m} \quad \text{LVS EN 1992-1-1 9.3.1.1 (3)}$$

$$s = \frac{A_{\varnothing}}{a_{s,1}} = \frac{0.0001131}{2.634 \cdot 10^{-5}} = 4.2937 \text{ m} > s_{max,slabs} = 0.25 \text{ m} \quad \text{!!} \rightarrow s = s_{max,slabs} = 0.25 \text{ m}$$

Longitudinal rebars: $\varnothing 12 \text{ mm}/250 \text{ mm}$

$$a_{s1,prov} = \frac{A_{\varnothing}}{s} = \frac{0.0001131}{0.25} = 452 \text{ mm}^2/\text{m} > a_{s,1} = 26 \text{ mm}^2/\text{m} \quad \checkmark$$

8.2.2.2. Investigation at the edge of the step - cross-section E and F

Tension reinforcement in y direction



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$$m_d = \text{Max}(m_{dE} ; m_{dF}) = \text{Max}(2.5595 ; 2.5595) = 2.5595 \text{ kNm/m}$$

$$d = h - u_{By} = 0.3 - 0.048 = 0.252 \text{ m}$$

$$x_{c0} = \xi_0 \cdot d = 0.49339 \cdot 0.252 = 0.12433 \text{ m}$$

$$x_c = d - \sqrt{d^2 - \frac{2 \cdot m_d}{f_{cd,eff}}} = 0.252 - \sqrt{0.252^2 - \frac{2 \cdot 2.5595}{20000}} = 0.00050835 \text{ m} < x_{c0} = \xi_0 \cdot d = 0.49339 \cdot 0.252 = 0.12433 \text{ m}$$

Tension reinforcement area:

The minimum area of longitudinal tension reinforcement: LVS EN 1992-1-1 9.2.2.1. (1) (9.1N)

$$a_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot d = 0.26 \cdot \frac{2.8965}{500} \cdot 0.252 = 0.00037955 \text{ m}^2/\text{m} > 0.0013 \cdot d = 0.0013 \cdot 0.252 = 0.0003276 \text{ m}^2/\text{m} = 380$$

mm²/m

$$a_{s,1} = \frac{x_c \cdot f_{cd,eff}}{f_{yd}} = \frac{0.00050835 \cdot 20000}{435000} = 2.3372 \cdot 10^{-5} \text{ m}^2/\text{m} = 23 \text{ mm}^2/\text{m} < a_{s,min} = 380 \text{ mm}^2/\text{m} \rightarrow a_{s,1} = a_{s,min} = 380$$

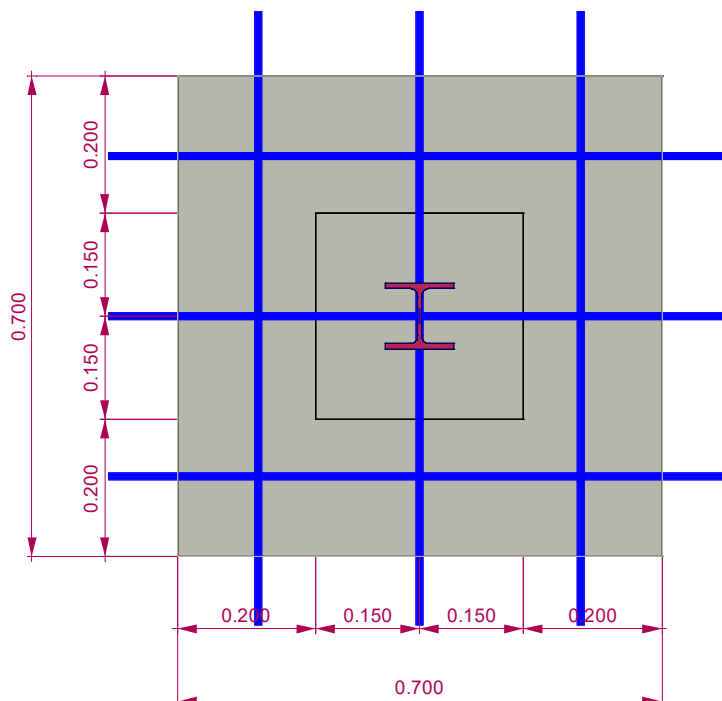
mm²/m

$$s_{max,slabs} = 2 \cdot h = 2 \cdot 0.3 = 0.6 \text{ m} > 0.25 \text{ m} \rightarrow s_{max,slabs} = 0.25 \text{ m} \quad \text{LVS EN 1992-1-1 9.3.1.1 (3)}$$

$$s = \frac{A_\varnothing}{a_{s,1}} = \frac{0.0001131}{0.00037955} = 0.29797 \text{ m} > s_{max,slabs} = 0.25 \text{ m} \quad \text{!!} \rightarrow s = s_{max,slabs} = 0.25 \text{ m}$$

Longitudinal rebars: Ø 12 mm/250 mm

$$a_{s1,prov} = \frac{A_\varnothing}{s} = \frac{0.0001131}{0.25} = 452 \text{ mm}^2/\text{m} > a_{s,1} = 380 \text{ mm}^2/\text{m} \quad \checkmark$$



9. Settlement evaluation

Stress-strain method LVS EN 1997-1 Annex F

9.1. Partial factors



Permanent, unfavourable actions	$\gamma_{G,unfav}$	1
Permanent, favourable actions	$\gamma_{G,fav}$	1
Variable, unfavourable actions	$\gamma_{Q,unfav}$	1
Variable, favourable actions	$\gamma_{Q,fav}$	0
Weight density	γ_γ	1

LVS EN 1997-12.4.8 (2) Values of partial factors for serviceability limit states should normally be taken as 1.0.

The effective overburden stress at the foundation base:

$$q' = \gamma_\gamma \cdot q'_k = 1 \cdot 10.928 = 10.928 \text{ kPa}$$

9.2. Design value of loads at the top of the footing - Nodal support internal forces

Load case: [Gk] {0.6*Qk} (SLS Quasipermanent)

$$F_x = 0.052383 \text{ kN} \quad F_y = 0 \text{ kN} \quad F_z = -20.846 \text{ kN} \quad M_x = 0 \text{ kNm} \quad M_y = -0.10635 \text{ kNm}$$

Vertical load: $V = -F_z = -(-20.846) = 20.846 \text{ kN} (\downarrow)$

9.3. Design value of loads at the base of footing

Design value of the weight of the footing: $G_d = G_k \cdot \gamma_{G,unfav} = 6.2268 \cdot 1 = 6.2268 \text{ kN} (\downarrow)$

$$H_{dx} = F_x = 0.052383 \text{ kN} \quad H_{dy} = F_y = 0 \text{ kN}$$

$$V_d = V + G_d = 20.846 + 6.2268 = 27.073 \text{ kN} (\downarrow)$$

Eccentricity of the vertical load (V_d) relative to the center of the footing:

$$e_x = \frac{M_y + F_x \cdot (h + h_b)}{V_d} = \frac{(-0.10635) + 0.052383 \cdot (0.3 + 0.08)}{27.073} = -0.00281 \text{ m}$$

$$e_y = 0 \text{ m}$$

Effective width of the footing:

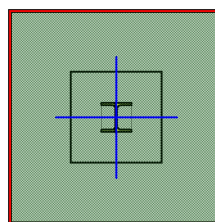
$$B' = \left(\frac{b_x}{2} - |e_x| \right) \cdot 2 = \left(\frac{0.7}{2} - |(-0.00281)| \right) \cdot 2 = 0.69438 \text{ m}$$

Effective length of the footing:

$$L' = \left(\frac{b_y}{2} - |e_y| \right) \cdot 2 = \left(\frac{0.7}{2} - |0| \right) \cdot 2 = 0.7 \text{ m}$$

Effective area of the footing:

$$A' = B' \cdot L' = 0.69438 \cdot 0.7 = 0.48607 \text{ m}^2$$



The normal stress below the corner of a rectangular loaded area at depth z is:

$$\sigma_z = \frac{p}{2 \cdot \pi} \cdot \left(\arctan \frac{b}{z} \cdot \frac{a \cdot (a^2 + b^2) - 2 \cdot a \cdot z \cdot (R - z)}{(a^2 + b^2) \cdot (R - z) - z \cdot (R - z)^2} + \frac{b \cdot z}{b^2 + z^2} \cdot \frac{a \cdot (R^2 + z^2)}{(a^2 + z^2) \cdot R} \right)$$

after Steinbrenner

where:

p is the uniformly distributed load at the loaded area

a and b are the length and width of the rectangular loaded area

$$R = \sqrt{a^2 + b^2 + z^2}$$

Stress under the characteristic point:



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$$\sigma_{z,k} = \sigma_{z,I} + \sigma_{z,II} + \sigma_{z,III} + \sigma_{z,IV}$$

	a	b
$\sigma_{z,I}$	$(0.5 - 0.37) \cdot L' = (0.5 - 0.37) \cdot 0.7 = 0.091 \text{ m}$	$(0.5 - 0.37) \cdot B' = (0.5 - 0.37) \cdot 0.69438 = 0.090269 \text{ m}$
$\sigma_{z,II}$	$(0.5 + 0.37) \cdot L' = (0.5 + 0.37) \cdot 0.7 = 0.609 \text{ m}$	$(0.5 - 0.37) \cdot B' = (0.5 - 0.37) \cdot 0.69438 = 0.090269 \text{ m}$
$\sigma_{z,III}$	$(0.5 + 0.37) \cdot L' = (0.5 + 0.37) \cdot 0.7 = 0.609 \text{ m}$	$(0.5 + 0.37) \cdot B' = (0.5 + 0.37) \cdot 0.69438 = 0.60411 \text{ m}$
$\sigma_{z,IV}$	$(0.5 + 0.37) \cdot B' = (0.5 + 0.37) \cdot 0.69438 = 0.60411 \text{ m}$	$(0.5 - 0.37) \cdot L' = (0.5 - 0.37) \cdot 0.7 = 0.091 \text{ m}$

Distance of the characteristic point from the central axes of the loaded area is $0.37 \cdot B'$ and $0.37 \cdot L'$.

The effective vertical stress due to the foundation load at the foundation base:

$$q_{E,d} = \frac{V_d}{A'} = \frac{27.073}{0.48607} = 55.697 \text{ kPa}$$

$$p = q_{E,d} - q' = 55.697 - 10.928 = 44.769 \text{ kPa}$$

The effective vertical stress due to the foundation load at the limit depth: $\sigma_{D_{lim}} = 6.7455 \text{ kPa}$

The effective overburden stress at the limit depth: $q_{D_{lim}} = 33.728 \text{ kPa}$

Limit depth: $D_{lim} = -1.5905 \text{ m}$ This depth may normally be taken as the depth at which the effective vertical stress due to the foundation load is 20 % of the effective overburden stress. LVS EN 1997-1 6.6.2 (6)

Settlement: $s = \Sigma s_i = 0.276 \text{ mm} < s_{lim} = 50.000 \text{ mm}$ **passed**

Reference soil layer: 19pc - ()

Oedometer modulus of the reference soil layer: $E_{oed,ref} = 65000 \text{ kPa}$

Stress in soil sublayers and settlement of sublayers

i in	z_0 [m]	h_i [m]	$h_{i,eq}$ [m]	σ_z [kPa]	q' [kPa]	σ_z/q' [%]	$\sigma_{z,i,avg}$ [kPa]	Δs_i [mm]	Σs_i [mm]
0.	0	0	0	0	0	—	0	0	0
1.	-0.1	0.1	0.031336	0	1.8639	0	0	0	0
2.	-0.2	0.1	0.031336	0	3.7278	0	0	0	0
3.	-0.3	0.1	0.031336	0	5.5917	0	0	0	0
4.	-0.4	0.1	0.031336	0	7.4556	0	0	0	0
5.	-0.5	0.1	0.031336	0	9.3195	0	0	0	0
6.	-0.55	0.05	0.015668	0	10.251	0	0	0	0
7.	-0.58	0.03	0.03	44.769	10.928	409.66	22.384	0	0
8.	-0.59	0.01	0.01	44.745	11.154	401.16	44.757	0.007	0.007
9.	-0.6	0.01	0.01	44.586	11.38	391.81	44.666	0.007	0.014
10.	-0.61	0.01	0.01	44.194	11.605	380.81	44.39	0.007	0.021
11.	-0.62	0.01	0.01	43.528	11.831	367.91	43.861	0.007	0.027
12.	-0.63	0.01	0.01	42.598	12.056	353.32	43.063	0.007	0.034
13.	-0.7	0.07	0.07	33.342	13.636	244.52	37.97	0.041	0.075
14.	-0.8	0.1	0.1	24.167	15.892	152.07	28.755	0.044	0.119
15.	-0.9	0.1	0.1	19.297	18.149	106.33	21.732	0.033	0.153
16.	-1	0.1	0.1	16.121	20.405	79.006	17.709	0.027	0.180
17.	-1.1	0.1	0.1	13.711	22.661	60.503	14.916	0.023	0.203



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18.	- 1.2	0.1	0.1	11.753	24.917	47.168	12.732	0.020	0.222
19.	- 1.3	0.1	0.1	10.125	27.174	37.259	10.939	0.017	0.239
20.	- 1.4	0.1	0.1	8.7607	29.43	29.768	9.4427	0.015	0.254
21.	- 1.5	0.1	0.1	7.616	31.686	24.036	8.1883	0.013	0.266
22.	- 1.5905	0.090473	0.090473	6.7455	33.728	20	7.1807	0	0.276
23.	- 1.6	0.009527	0.009527	6.6539	33.943	19.603	6.6997	0.011	0.277
24.	- 1.7	0.1	0.1	5.8434	36.199	16.142	6.2486	0.010	0.287
25.	- 1.8	0.1	0.1	5.1585	38.455	13.414	5.5009	0.008	0.295
26.	- 1.85	0.05	0.05	4.8561	39.583	12.268	5.0073	0.004	0.299

where:

 z_0 : Depth h_i : Thickness of the soil layer $h_{i,eq}$: Equivalent thickness

$$h_{i,eq} = h_i \cdot \left(\frac{E_{oed,i}}{E_{oed,ref}} \right)^{1/3} \quad \text{Thenn de Barros}$$

where:

 $E_{oed,i}$: Oedometer modulus of the soil layer $E_{oed,ref}$: Oedometer modulus of the reference soil layer σ_z : The effective vertical stress due to the foundation load q' : The effective overburden stress s_i : Settlement of the soil layer

$$s_i = \frac{\sigma_{z,i,avg}}{E_{oed,i}} \cdot h_i$$

 Σs_i : Total settlement at the given depth



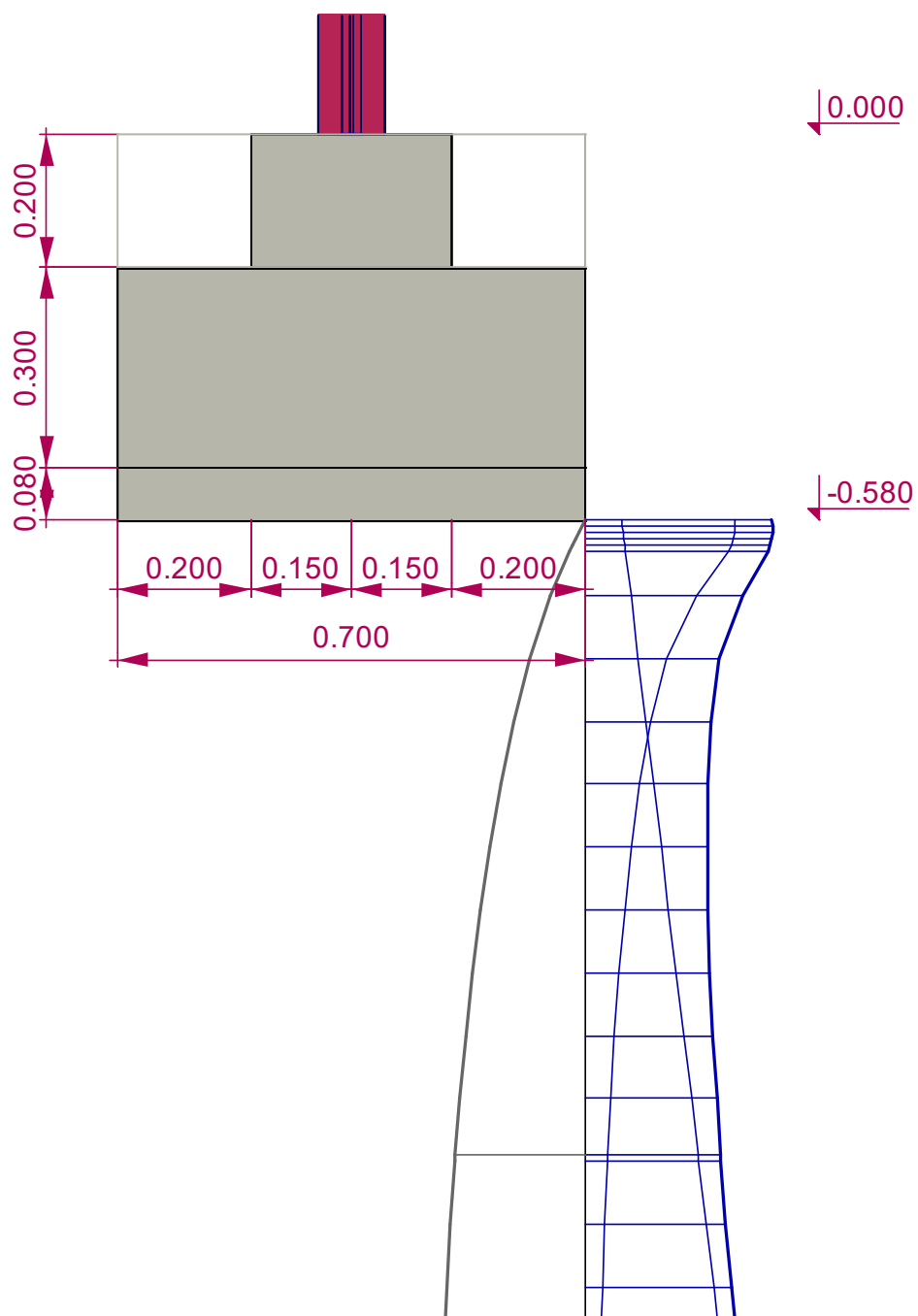
Magnētiskās rezonanses iekārtas uzstādīšana Slimnīcas ielā 25, Lie

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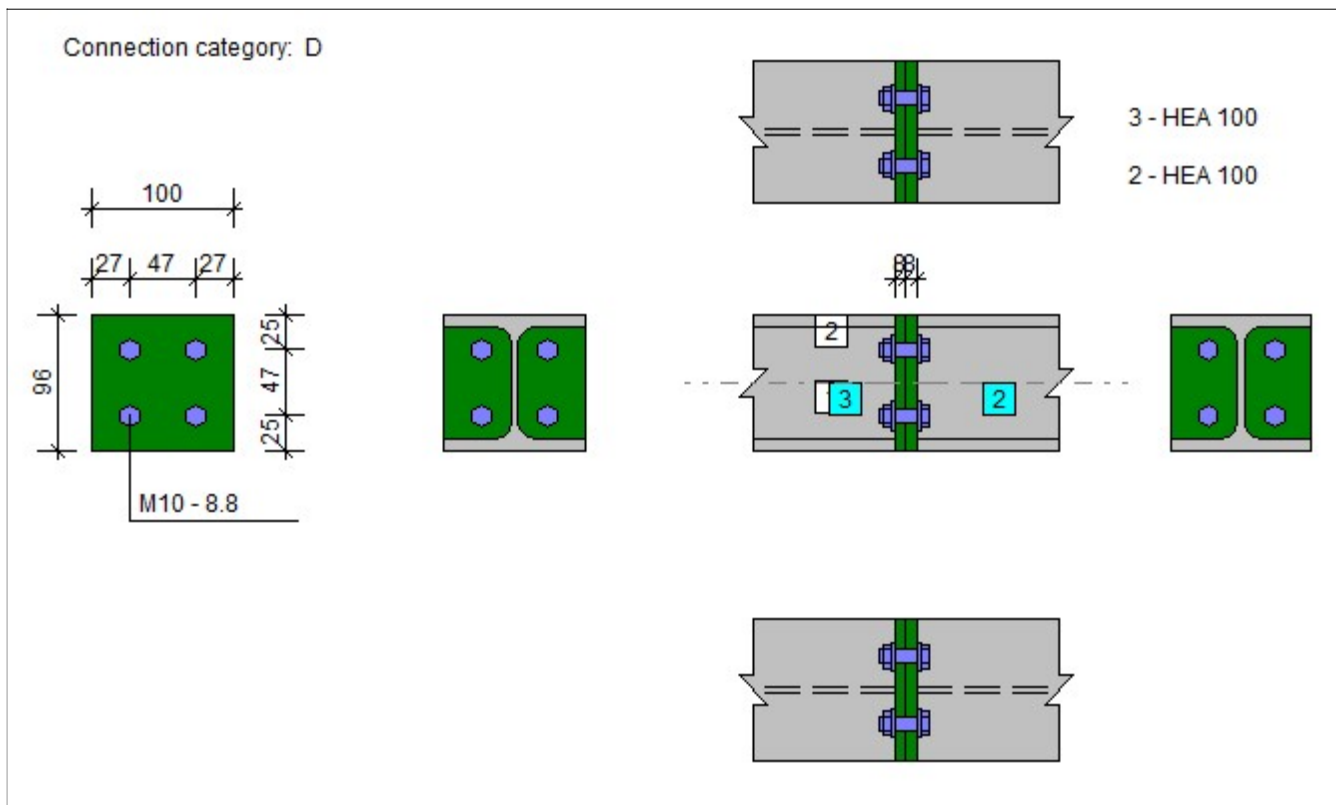
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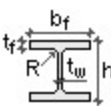
	Beam - beam (front plate) EN 1993-1-8:2006	Ratio : 0.069	
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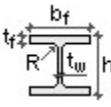
Structure node: 9

Structure members: 3, 2



Data

Left beam HEA 100					
	h_b	b_{fb}	t_{fb}	t_{wb}	R_b
	96.00mm	100.00mm	8.00mm	5.00mm	12.00mm
	A_b	J_{y0b}	J_{z0b}	y_{0b}	z_{0b}
	21.24cm ²	349.24cm ⁴	133.81cm ⁴	50.00mm	48.00mm
Material	Grade	f_y	f_u		
	S 355	355.00MPa	490.00MPa		

Right beam HEA 100					
	h_b	b_{fb}	t_{fb}	t_{wb}	R_b
	96.00mm	100.00mm	8.00mm	5.00mm	12.00mm
	A_b	J_{y0b}	J_{z0b}	y_{0b}	z_{0b}
	21.24cm ²	349.24cm ⁴	133.81cm ⁴	50.00mm	48.00mm



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Material	Grade	f_y	f_u
	S 355	355.00MPa	490.00MPa

Front plate			
	l_p	h_p	t_p
	100.00mm	96.00mm	8.00mm
Material	Grade	f_y	f_u
	S 355	355.00MPa	490.00MPa

Bolts connecting front plates

Grade		8.8
Yield point	$f_y =$	640.00MPa
Resistance on tension	$f_u =$	800.00MPa
Bolt diameter	$d =$	10.00mm
Bolt opening diameter	$d_0 =$	11.00mm
Area of bolt section	$A =$	0.79cm ²
Effective section area of a bolt	$A_s =$	0.58cm ²
Number of rows	$w =$	2.00mm
Distance from horizontal edge	$e_1 =$	24.50mm
Horizontal spacing	$w_1 =$	47.00mm
Number of bolts in rows		$m_1 = 2m_2 = 2$
Vertical spacing of rows		$p_1 = 47.00mm$

Welds

Left side

Thickness of butt welds connecting beam web and front plate	$a_w =$	4.00mm
Thickness of butt welds connecting beam flanges and front plate	$a_{fL} =$	4.00mm

Right side

Thickness of butt welds connecting beam web and front plate	$a_w =$	4.00mm
Thickness of butt welds connecting beam flanges and front plate	$a_{fL} =$	4.00mm

Warnings

Forces

Design loads [1.15*Gk+1.50*Qk]



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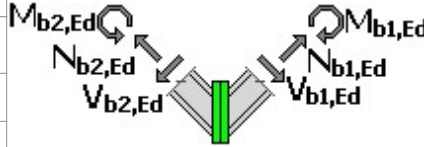
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Left beam		
Axial force	$N_{b2,Ed} =$	$-49.22kN$
Shear force	$V_{b2,Ed} =$	$0.00kN$
Bending moment	$M_{b2,Ed} =$	$0.00kNm$
Right beam		
Axial force	$N_{b1,Ed} =$	$-49.22kN$
Shear force	$V_{b1,Ed} =$	$0.00kN$
Bending moment	$M_{b1,Ed} =$	$0.00kNm$



Results

Left side

Bolts connecting front plates

Tensile resistance of a bolt

$$F_{t,Rd} = \frac{k_z \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.900 \cdot 800.00MPa \cdot 0.58cm^2}{1.250} = 33.41kN$$

Shear section area of bolt

$$A = 0.25 \cdot \pi \cdot d^2 = 0.25 \cdot \pi \cdot (10.00mm)^2 = 0.79cm^2$$

Shear resistance of bolt in one surface

$$F_{v,Rd} = \frac{\alpha_v \cdot m \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0.600 \cdot 1 \cdot 800.00MPa \cdot 0.79cm^2}{1.250} = 30.16kN$$

Punching shear resistance of a bolt

$$B_{p,Rd} = \frac{0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_{up}}{\gamma_{M2}} = \frac{0.6 \cdot \pi \cdot 16.88mm \cdot 8.00mm \cdot 490.00MPa}{1.250} = 99.81kN$$

Beam flange and web in compression

Plastic section modulus

$$W_{pl} = 83.02cm^3$$

The design resistance for bending of cross-section

$$M_{c,Rd} = \frac{W_{pl} \cdot f_{yb}}{\gamma_{M0}} = \frac{83.02cm^3 \cdot 355.00MPa}{1.000} = 29.47kNm$$

Distance between beam flanges

$$h_f = 88.00mm$$



Design resistance of flange beam subject compression

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h_f} = \frac{29.47kNm}{88.00mm} = 334.90kN$$

Tension zone

BOLTS ROW 1

End plate in bending

Geometrical parameters

Bolt distance from outer edge

$$e_{ep} = 26.50mm$$

Bolt distance from beam web

$$m_{ep} = 0.5 \cdot (w - t_{wb}) - 0.8 \cdot \sqrt{2} \cdot a_w = 0.5 \cdot (47.00mm - 5.00mm) - 0.8 \cdot \sqrt{2} \cdot 4.00mm = 16.47mm$$

Distance $e_{\min}$

$$e_{\min} = 26.50mm$$

Calculation parameter n

$$n = \min(e_{\min}; 1.25 \cdot m_{ep}) = \min(26.50mm; 1.25 \cdot 16.47mm) = 20.59mm$$

Calculation parameter

$$m_{2ep} = p_1 + e_1 - e_{p1} - t_{fb} - 0.8 \cdot a_f \cdot \sqrt{2} = 47.00mm + 24.50mm - 0.00mm - 8.00mm - 0.8 \cdot 4.00mm \cdot \sqrt{2} = 11.97mm$$

$$\lambda_1 = \frac{m_{ep}}{m_{ep} + e_{ep}} = \frac{16.47mm}{16.47mm + 26.50mm} = 0.383$$

$$\lambda_2 = \frac{m_{2ep}}{m_{ep} + e_{ep}} = \frac{11.97mm}{16.47mm + 26.50mm} = 0.279$$

$$\alpha = 7.315$$

Effective length for one bolt in circular patterns

$$l_{eff,cp} = 2 \cdot \pi \cdot m_{ep} = 2 \cdot \pi \cdot 16.47mm = 103.51mm$$

Effective length for one bolt in non-circular patterns

$$l_{eff,nc} = \alpha \cdot m_{ep} = 7.315 \cdot 16.47mm = 120.51mm$$

Effective length for one bolt for 1 mode

$$l_{eff,1} = \min(l_{eff,cp}; l_{eff,nc}) = \min(103.51mm; 120.51mm) = 103.51mm$$

Effective length for one bolt for 2 mode

$$l_{eff,2} = l_{eff,nc} = 120.51mm$$

Model 1: Complete yielding of the end plate

$$M_{pl,1,Rd} = \frac{0.25 \cdot l_{eff,1} \cdot t_p^2 \cdot f_{yp}}{Y_{M0}} = \frac{0.25 \cdot 103.51mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 0.59kNm$$

Method 1

$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 0.59kNm}{16.47mm} = 142.75kN$$

Method 2 (alternative method)

Parameter for bearing zone

$$e_w = 0.25 \cdot d_w = 0.25 \cdot 20.00mm = 5.00mm$$

$$F_{T,1,Rd2} = \frac{(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{2 \cdot m_{ep} \cdot n - e_w \cdot (m_{ep} + n)} = \frac{(8 \cdot 20.59mm - 2 \cdot 5.00mm) \cdot 0.59kNm}{2 \cdot 16.47mm \cdot 20.59mm - 5.00mm \cdot (16.47mm + 20.59mm)} = 184.48kN$$

$$F_{T,1,Rd} = \min(F_{T,1,Rd1}; F_{T,1,Rd2}) = \min(142.75kN; 184.48kN) = 142.75kN$$

Model 2: Bolt failure with yielding of the end plate

$$M_{pl,2,Rd} = \frac{0.25 \cdot l_{eff,2} \cdot t_p^2 \cdot f_{yp}}{Y_{M0}} = \frac{0.25 \cdot 120.51mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 0.68kNm$$

$$F_{T,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \cdot \sum F_{t,Rd}}{m_{ep} + n} = \frac{2 \cdot 0.68kNm + 20.59mm \cdot 2 \cdot 33.41kN}{16.47mm + 20.59mm} = 74.05kN$$

Model 3: Bolt failure

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 33.41kN = 66.82kN$$

Component resistance

$$F_{t,ep,Rd(1)} = \min(F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}) = \min(142.75kN; 74.05kN; 66.82kN) = 66.82kN$$

Beam web in tension

Effective width of beam web in tension

$$b_{eff,t,wb} = l_{eff1(1)} = 103.51mm$$

Component resistance



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$$F_{t,wb,Rd(1)} = \frac{b_{eff,t,wb} \cdot t_{wb} \cdot f_{yb}}{\gamma_{M0}} = \frac{103.51mm \cdot 5.00mm \cdot 355.00MPa}{1.000} = 183.73kN$$

Resistance of the bolt row 1

$F_{t,Rd(1)M} = \min$		
·	$F_{t,ep,Rd(1)} =$	66.82kN
·	$F_{t,wb,Rd(1)} =$	183.73kN
·	$F_{cfb,Rd(1)} =$	334.90kN
		= 66.82kN

BOLTS ROW 2

End plate in bending

Geometrical parameters

Bolt distance from outer edge

$$e_{ep} = 26.50mm$$

Bolt distance from beam web

$$m_{ep} = 0.5 \cdot (w - t_{wb}) - 0.8 \cdot \sqrt{2} \cdot a_w = 0.5 \cdot (47.00mm - 5.00mm) - 0.8 \cdot \sqrt{2} \cdot 4.00mm = 16.47mm$$

Distance e{min}

$$e_{min} = 26.50mm$$

Calculation parameter n

$$n = \min(e_{min}; 1.25 \cdot m_{ep}) = \min(26.50mm; 1.25 \cdot 16.47mm) = 20.59mm$$

Effective length for one bolt in circular patterns

$$l_{eff,cp} = 2 \cdot \pi \cdot m_{ep} = 2 \cdot \pi \cdot 16.47mm = 103.51mm$$

Effective length for one bolt in non-circular patterns

$$l_{eff,nc} = 4 \cdot m_{ep} + 1.25 \cdot e = 4 \cdot 16.47mm + 1.25 \cdot 26.50mm = 99.02mm$$

Effective length for one bolt for 1 mode

$$l_{eff,1} = \min(l_{eff,cp}; l_{eff,nc}) = \min(103.51mm; 99.02mm) = 99.02mm$$

Effective length for one bolt for 2 mode

$$l_{eff,2} = l_{eff,nc} = 99.02mm$$

Model 1: Complete yielding of the end plate

$$M_{pl,1,Rd} = \frac{0.25 \cdot l_{eff,1} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 99.02mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 0.56kNm$$

Method 1

$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 0.56kNm}{16.47mm} = 136.56kN$$

Method 2 (alternative method)

Parameter for bearing zone

$$e_w = 0.25 \cdot d_w = 0.25 \cdot 20.00mm = 5.00mm$$

$$F_{T,1,Rd2} = \frac{(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{2 \cdot m_{ep} \cdot n - e_w \cdot (m_{ep} + n)} = \frac{(8 \cdot 20.59mm - 2 \cdot 5.00mm) \cdot 0.56kNm}{2 \cdot 16.47mm \cdot 20.59mm - 5.00mm \cdot (16.47mm + 20.59mm)} = 176.48kN$$

$$F_{T,1,Rd} = \min(F_{T,1,Rd1}; F_{T,1,Rd2}) = \min(136.56kN; 176.48kN) = 136.56kN$$

Model 2: Bolt failure with yielding of the end plate

$$M_{pl,2,Rd} = \frac{0.25 \cdot l_{eff,2} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 99.02mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 0.56kNm$$

$$F_{T,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \cdot \sum F_{t,Rd}}{m_{ep} + n} = \frac{2 \cdot 0.56kNm + 20.59mm \cdot 2 \cdot 33.41kN}{16.47mm + 20.59mm} = 67.47kN$$

Model 3: Bolt failure

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 33.41kN = 66.82kN$$

Component resistance

$$F_{t,ep,Rd(1)} = \min(F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}) = \min(136.56kN; 67.47kN; 66.82kN) = 66.82kN$$

Beam web in tension

Effective width of beam web in tension

$$b_{eff,t,wb} = l_{eff1(2)} = 99.02mm$$

Component resistance

$$F_{t,wb,Rd(2)} = \frac{b_{eff,t,wb} \cdot t_{wb} \cdot f_{yb}}{\gamma_{M0}} = \frac{99.02mm \cdot 5.00mm \cdot 355.00MPa}{1.000} = 175.77kN$$

End plate in bending

Row 1+2 considered as group

Circular patterns

Bolts row 1

$$l_{eff,cp(1,g)} = \pi \cdot m_{ep} + p = \pi \cdot 16.47mm + 47.00mm = 98.76mm$$

Bolts row 2

$$l_{eff,cp(2,g)} = \pi \cdot m_{ep} + p = \pi \cdot 16.47mm + 47.00mm = 98.76mm$$

$$\sum l_{eff(1+2)} = l_{eff,cp(1,g)} + l_{eff,cp(2,g)} = 98.76mm + 98.76mm = 197.51mm$$

Non-circular patterns

Bolts row 1

$$\begin{aligned} l_{eff,nc(1,g)} &= 0.5 \cdot p + \alpha \cdot m_{ep} - (2 \cdot m_{ep} + 0.625 \cdot e) = \\ &= 0.5 \cdot 47.00mm + 7.315 \cdot 16.47mm - (2 \cdot 16.47mm + 0.625 \cdot 26.50mm) = 94.49mm \end{aligned}$$

Bolts row 2

$$l_{eff,nc(2,g)} = 2 \cdot m_{ep} + 0.625 \cdot e + 0.5 \cdot p = 2 \cdot 16.47mm + 0.625 \cdot 26.50mm + 0.5 \cdot 47.00mm = 73.01mm$$

$$\sum l_{eff,nc(1+2)} = l_{eff,nc(1,g)} + l_{eff,nc(2,g)} = 94.49mm + 73.01mm = 167.51mm$$

Model 1: Complete yielding of the end plate

Effective length for one bolt for 1 mode

$$\sum l_{eff,1(1+2)} = \min(\sum l_{eff,cp(1+2)}; \sum l_{eff,nc(1+2)}) = \min(197.51mm; 167.51mm) = 167.51mm$$

$$M_{pl,1,Rd} = \frac{0.25 \cdot \sum l_{eff,1(1+2)} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 167.51mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 1.12kNm$$

Method 1

$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 1.12kNm}{16.47mm} = 272.39kN$$

$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 1.12kNm}{16.47mm} = 272.39kN$$

Method 2 (alternative method)

$$F_{T,1,Rd2} = \frac{(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{2 \cdot m_{ep} \cdot n - e_w \cdot (m_{ep} + n)} = \frac{(8 \cdot 20.59mm - 2 \cdot 5.00mm) \cdot 1.12kNm}{2 \cdot 16.47mm \cdot 20.59mm - 5.00mm \cdot (16.47mm + 20.59mm)} = 352.01kN$$



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$$F_{T,1,Rd2} = \frac{(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{2 \cdot m_{ep} \cdot n - e_w \cdot (m_{ep} + n)} = \frac{(8 \cdot 20.59mm - 2 \cdot 5.00mm) \cdot 1.12kNm}{2 \cdot 16.47mm \cdot 20.59mm - 5.00mm \cdot (16.47mm + 20.59mm)} = 352.01kN$$

$$F_{T,1,Rd} = \min(F_{T,1,Rd1}; F_{T,1,Rd2}) = \min(272.39kN; 352.01kN) = 272.39kN$$

Model 2: Bolt failure with yielding of the end plate

Effective length for one bolt for 2 mode

$$\sum l_{eff,2(1+2)} = \sum l_{eff,nc(1+2)} = 167.51mm$$

$$M_{pl,2,Rd} = \frac{0.25 \cdot \sum l_{eff,2(1+2)} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 167.51mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 1.12kNm$$

$$F_{T,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \cdot \sum B_{p,Rd}}{m_{ep} + n} = \frac{2 \cdot 1.12kNm + 20.59mm \cdot 4 \cdot 33.41kN}{16.47mm + 20.59mm} = 134.77kN$$

Model 3: Bolt failure

$$F_{T,3,Rd} = \sum F_{t,Rd} = 4 \cdot 33.41kN = 133.63kN$$

Resistance of group

$$F_{t,ep(1+2)} = \min(F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}) = \min(272.39kN; 134.77kN; 133.63kN) = 133.63kN$$

Beam web in tension

Effective width of beam web in tension

$$b_{eff,t,wb} = \sum l_{eff1(1+2)} = 197.51mm$$

Component resistance

$$F_{t,wb,Rd(1+2)} = \frac{b_{eff,t,wb} \cdot t_{wb} \cdot f_{yb}}{\gamma_{M0}} = \frac{197.51mm \cdot 5.00mm \cdot 355.00MPa}{1.000} = 350.58kN$$

Resistance of the bolt row 2

$F_{t,Rd(2)M} = \min$		
· $F_{t,ep,Rd(2)} =$	66.82kN	
· $F_{t,wb,Rd(2)} =$	175.77kN	
· $F_{t,ep,Rd(1+2)} - F_{t,Rd(1)} =$	$133.63kN - 66.82kN =$	66.82kN
· $F_{t,wb,Rd(1+2)} - F_{t,Rd(1)} =$	$350.58kN - 66.82kN =$	283.77kN
· $F_{cfb,Rd} - F_{t,Rd(1)} =$	$334.90kN - 66.82kN =$	268.08kN
$= 66.82kN$		

Bending resistance



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Real bending moment

$$M_0 = M_{b2,Ed} = 0.00kNm$$

The design moment resistance of the joint, assuming no axial force

$$M_{j,Rd} = F_{t,Rd(1)} \cdot h_1 + F_{t,Rd(2)} \cdot h_2 = 66.82kN \cdot 67.50mm + 66.82kN \cdot 20.50mm = 5.88kNm$$

$\frac{ M_0 }{M_{j,Rd}} \leq 1$	0.000 < 1.000	0.000	✓
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Shear resistance

Shear force

$$V_0 = -N_{b2,Ed} \cdot \sin(\alpha_2) + V_{b2,Ed} \cdot \cos(\alpha_2) = -((-49.22kN)) \cdot \sin(0.00Deg) + (0.00kN) \cdot \cos(0.00Deg) = 0.00kN$$

BOLTS ROW 1

Bolt bearing on the plate

Coefficient determined by bolt spacing

$$\alpha_b = \min(1.0; \frac{f_{ub}}{f_{up}}; \frac{e_1}{3 \cdot d_0}) = \min(1.0; \frac{800.00MPa}{490.00MPa}; \frac{24.50mm}{3 \cdot 11.00mm}) = 0.742$$

Coefficient determined by bolt spacing

$$k_1 = \min(2.5; 2.8 \cdot \frac{e_2}{d_0} - 1.7) = \min(2.5; 2.8 \cdot \frac{26.50mm}{11.00mm} - 1.7) = 2.500$$

Bearing resistance of a bolt

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p}{\gamma_{M2}} = \frac{2.500 \cdot 0.742 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 58.21kN$$

Resistance of the bolt row 1

$$V_{Rd(1)} = m_1 \cdot \min(F_{b,Rd}; F_{v,Rd}) = 2 \cdot \min(58.21kN; 30.16kN) = 60.32kN$$

BOLTS ROW 2

Bolt bearing on the plate

Coefficient determined by bolt spacing

$$\alpha_b = \min(1.0; \frac{f_{ub}}{f_{up}}; \frac{e_1}{3 \cdot d_0}) = \min(1.0; \frac{800.00MPa}{490.00MPa}; \frac{24.50mm}{3 \cdot 11.00mm}) = 0.742$$



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Coefficient determined by bolt spacing

$$k_1 = \min(2.5; 2.8 \cdot \frac{e_2}{d_0} - 1.7) = \min(2.5; 2.8 \cdot \frac{26.50mm}{11.00mm} - 1.7) = 2.500$$

Bearing resistance of a bolt

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p}{\gamma_{M2}} = \frac{2.500 \cdot 0.742 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 58.21kN$$

Resistance of the bolt row 2

$$V_{Rd(2)} = m_2 \cdot \min(F_{b,Rd}; F_{v,Rd}) = 2 \cdot \min(58.21kN; 30.16kN) = 60.32kN$$

$$V_{j,Rd} = V_{Rd(1)} + V_{Rd(2)} = 60.32kN + 60.32kN = 120.64kN$$

$\frac{ V_0 }{V_{j,Rd}} \leq 1$	$ 0.00kN < 120.64kN$	0.000	✓
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Butt welds connecting beam and front plate

Forces in welds

Axial force

$$N_0 = N_{b2,Ed} \cdot \cos(\alpha_2) + V_{b2,Ed} \cdot \sin(\alpha_2) = (-49.22kN) \cdot \cos(0.00Deg) + (0.00kN) \cdot \sin(0.00Deg) = -49.22kN$$

Shear force

$$V_0 = -N_{b2,Ed} \cdot \sin(\alpha_2) + V_{b2,Ed} \cdot \cos(\alpha_2) = -((-49.22kN)) \cdot \sin(0.00Deg) + (0.00kN) \cdot \cos(0.00Deg) = 0.00kN$$

Real bending moment

$$M_0 = M_{b2,Ed} = 0.00kNm$$

Geometric properties of welds

Beam

Area of horizontal welds on upper flange

$$A_{wfu} = [2 \cdot (b_{fb} - r_b) - t_{wb}] \cdot a_f = [2 \cdot (100.00mm - 12.00mm) - 5.00mm] \cdot 4.00mm = 6.84cm^2$$

Area of horizontal welds on lower flange

$$A_{wfl} = [2 \cdot (b_{fb} - r_b) - t_{wb}] \cdot a_f = [2 \cdot (100.00mm - 12.00mm) - 5.00mm] \cdot 4.00mm = 8.97cm^2$$

Area of vertical welds

$$A_{ww} = 2 \cdot \frac{h_b - 2 \cdot (t_{fb} + r_b)}{\cos(\alpha)} \cdot a_w = 2 \cdot \frac{96.00mm - 2 \cdot (8.00mm + 12.00mm)}{\cos(0.00Deg)} \cdot 4.00mm = 4.48cm^2$$

Area of all welds

$$A_w = A_{wfu} + A_{wfl} + A_{ww} = 6.84cm^2 + 8.97cm^2 + 4.48cm^2 = 20.29cm^2$$

Offset of welds centroid with respect to beam centroid



Magnētiskās rezonanses iekārtas uzstādīšana Slimnīcas ielā 25, Lie

Analysis by SCW constructions

 Model: **Pastiprinājums.axs**

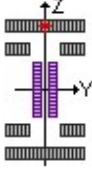
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$$e_{0w} = -18.28mm$$

Moment of inertia welds

$$I_w = 237.83cm^4$$

The point where the stresses are checked	$z_i = 66.28mm$
Elastic modulus of welds	
$W_w = 35.88cm^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.22kN}{20.29cm^2} = -24.26MPa$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{I_w} = \frac{0.00kNm \cdot 66.28mm}{237.83cm^4} = 0.00MPa$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -24.26MPa + (0.00MPa) = -24.26MPa$	
Normal perpendicular stress	
$\sigma_{\perp} = \sigma = -24.26MPa$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -24.26MPa < 352.80MPa$	0.069	✓
---	---------------------------	-------	---

$\sqrt{\sigma_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$24.26MPa < 435.56MPa$	0.056	✓
--	------------------------	-------	---

The point where the stresses are checked	$z_i = 46.28mm$
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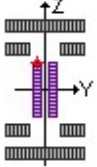
Magnētiskās rezonanses iekārtas uzstādīšana Slimnīcas ielā 25, Lie

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Elastic modulus of welds	
$W_w = 51.39cm^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.22kN}{20.29cm^2} = -24.26MPa$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{I_w} = \frac{0.00kNm \cdot 46.28mm}{237.83cm^4} = 0.00MPa$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -24.26MPa + (0.00MPa) = -24.26MPa$	
Normal perpendicular stress	
$\sigma_{\perp} = \sigma = -24.26MPa$	
Parallel tangent stress	
$\tau_{ } = \frac{V_0}{A_{ww}} = \frac{0.00kN}{4.48cm^2} = 0.00MPa$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -24.26MPa < 352.80MPa$	0.069	✓
---	---------------------------	-------	---

$\sqrt{\sigma_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$24.26MPa < 435.56MPa$	0.056	✓
--	------------------------	-------	---

The point where the stresses are checked	$z_i = -9.72mm$
--	-----------------



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Elastic modulus of welds	
$W_w = 244.66\text{cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.22\text{kN}}{20.29\text{cm}^2} = -24.26\text{MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{I_w} = \frac{0.00\text{kNm} \cdot (-9.72\text{mm})}{237.83\text{cm}^4} = 0.00\text{MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -24.26\text{MPa} + 0.00\text{MPa} = -24.26\text{MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \sigma = -24.26\text{MPa}$	
Parallel tangent stress	
$\tau_{\parallel} = \frac{V_0}{A_{ww}} = \frac{0.00\text{kN}}{4.48\text{cm}^2} = 0.00\text{MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -24.26\text{MPa} < 352.80\text{MPa}$	0.069	✓
---	---	-------	---

$\sqrt{\sigma_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$24.26\text{MPa} < 435.56\text{MPa}$	0.056	✓
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The point where the stresses are checked	$z_i = -29.72\text{mm}$
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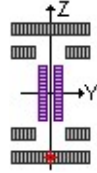
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Elastic modulus of welds	
$W_w = 80.02 \text{ cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.22 \text{ kN}}{20.29 \text{ cm}^2} = -24.26 \text{ MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{I_w} = \frac{0.00 \text{ kNm} \cdot (-29.72 \text{ mm})}{237.83 \text{ cm}^4} = 0.00 \text{ MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -24.26 \text{ MPa} + 0.00 \text{ MPa} = -24.26 \text{ MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \sigma = -24.26 \text{ MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -24.26 \text{ MPa} < 352.80 \text{ MPa}$	0.069	✓
---	---	-------	---

$\sqrt{\sigma_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$24.26 \text{ MPa} < 435.56 \text{ MPa}$	0.056	✓
--	--	-------	---

The rotational stiffness of a joint

The bolt elongation length

$$L_b = 2 \cdot t_p + 0.5 \cdot (m + k) + 2 \cdot t_{wa} = 2 \cdot 8.00 \text{ mm} + 0.5 \cdot (9.50 \text{ mm} + 6.40 \text{ mm}) + 2 \cdot 2.00 \text{ mm} = 27.95 \text{ mm}$$

$$k_{10} = \frac{3.2 \cdot A_s}{L_b} = \frac{3.2 \cdot 0.58 \text{ cm}^2}{27.95 \text{ mm}} = 3.32 \text{ mm}$$

Bolts row 1

End plate in bending

$$k_{5,1} = \frac{0.9 \cdot l_{\text{eff}} \cdot t_p^3}{m_{\text{ep}}^3} = \frac{0.9 \cdot 103.51 \text{ mm} \cdot (8.00 \text{ mm})^3}{(16.47 \text{ mm})^3} = 10.67 \text{ mm}$$



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$$k_{eff,1} = \frac{1}{\frac{1}{k_5} + \frac{1}{k_{10}}} = \frac{1}{\frac{1}{10.67mm} + \frac{1}{3.32mm}} = 2.53mm$$

Bolts row 2

End plate in bending

$$k_{5,2} = \frac{0.9 \cdot l_{eff} \cdot t_p^3}{m_{ep}^3} = \frac{0.9 \cdot 99.02mm \cdot (8.00mm)^3}{(16.47mm)^3} = 10.20mm$$

$$k_{eff,2} = \frac{1}{\frac{1}{k_5} + \frac{1}{k_{10}}} = \frac{1}{\frac{1}{10.20mm} + \frac{1}{3.32mm}} = 2.51mm$$

Lever arm of internal forces

$$z_{eq} = \frac{k_{eff,1} \cdot h_1^2 + k_{eff,2} \cdot h_2^2}{k_{eff,1} \cdot h_1 + k_{eff,2} \cdot h_2} = \frac{2.53mm \cdot (67.50mm)^2 + 2.51mm \cdot (20.50mm)^2}{2.53mm \cdot 67.50mm + 2.51mm \cdot 20.50mm} = 56.64mm$$

Equivalent stiffness coefficient

$$k_{eq} = \frac{k_{eff,1} \cdot h_1 + k_{eff,2} \cdot h_2}{z_{eq}} = \frac{2.53mm \cdot 67.50mm + 2.51mm \cdot 20.50mm}{56.64mm} = 3.92mm$$

The initial rotational stiffness of a joint

$$S_{j,ini} = \frac{\frac{E \cdot z_{eq}^2}{1}}{\frac{1}{k_{eq}}} = \frac{210000.00MPa \cdot (56.64mm)^2}{3.92mm} = 2643.84 \frac{kNm}{rad}$$

The rotational stiffness of a pinned joint

$$S_{j,pin} = \frac{0.5 \cdot E \cdot I_{yb}}{L_b} = \frac{0.5 \cdot 210000.00MPa \cdot 349.24cm^4}{100.00mm} = 3666.98 \frac{kNm}{rad}$$

The rotational stiffness of a rigid joint

$$S_{j,rig} = \frac{k_b \cdot E \cdot I_{yb}}{L_b} = \frac{8.000 \cdot 210000.00MPa \cdot 349.24cm^4}{100.00mm} = 58671.72 \frac{kNm}{rad}$$

Stiffness ratio

$$\mu = 1.000$$

The rotational stiffness of a joint

$$S_j = \frac{S_{j,ini}}{\mu} = \frac{2643.84 \frac{kNm}{rad}}{1.000} = 2643.84 \frac{kNm}{rad}$$



Classification of joint

Nominally pinned

Right side

Bolts connecting front plates

Tensile resistance of a bolt

$$F_{t,Rd} = \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.900 \cdot 800.00 \text{ MPa} \cdot 0.58 \text{ cm}^2}{1.250} = 33.41 \text{ kN}$$

Shear section area of bolt

$$A = 0.25 \cdot \pi \cdot d^2 = 0.25 \cdot \pi \cdot (10.00 \text{ mm})^2 = 0.79 \text{ cm}^2$$

Shear resistance of bolt in one surface

$$F_{v,Rd} = \frac{\alpha_v \cdot m \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0.600 \cdot 1 \cdot 800.00 \text{ MPa} \cdot 0.79 \text{ cm}^2}{1.250} = 30.16 \text{ kN}$$

Punching shear resistance of a bolt

$$B_{p,Rd} = \frac{0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_{up}}{\gamma_{M2}} = \frac{0.6 \cdot \pi \cdot 16.88 \text{ mm} \cdot 8.00 \text{ mm} \cdot 490.00 \text{ MPa}}{1.250} = 99.81 \text{ kN}$$

Beam flange and web in compression

Plastic section modulus

$$W_{pl} = 83.02 \text{ cm}^3$$

The design resistance for bending of cross-section

$$M_{c,Rd} = \frac{W_{pl} \cdot f_{yb}}{\gamma_{M0}} = \frac{83.02 \text{ cm}^3 \cdot 355.00 \text{ MPa}}{1.000} = 29.47 \text{ kNm}$$

Distance between beam flanges

$$h_f = 88.00 \text{ mm}$$

Design resistance of flange beam subject compression

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h_f} = \frac{29.47 \text{ kNm}}{88.00 \text{ mm}} = 334.90 \text{ kN}$$

Tension zone

BOLTS ROW 1

End plate in bending

Geometrical parameters

Bolt distance from outer edge

$$e_{ep} = 26.50mm$$

Bolt distance from beam web

$$m_{ep} = 0.5 \cdot (w - t_{wb}) - 0.8 \cdot \sqrt{2} \cdot a_w = 0.5 \cdot (47.00mm - 5.00mm) - 0.8 \cdot \sqrt{2} \cdot 4.00mm = 16.47mm$$

Distance $e_{\min}$

$$e_{\min} = 26.50mm$$

Calculation parameter n

$$n = \min(e_{\min}; 1.25 \cdot m_{ep}) = \min(26.50mm; 1.25 \cdot 16.47mm) = 20.59mm$$

Calculation parameter

$$m_{2ep} = p_1 + e_1 - e_{p1} - t_{fb} - 0.8 \cdot a_f \cdot \sqrt{2} = 47.00mm + 24.50mm - 0.00mm - 8.00mm - 0.8 \cdot 4.00mm \cdot \sqrt{2} = 11.97mm$$

$$\lambda_1 = \frac{m_{ep}}{m_{ep} + e_{ep}} = \frac{16.47mm}{16.47mm + 26.50mm} = 0.383$$

$$\lambda_2 = \frac{m_{2ep}}{m_{ep} + e_{ep}} = \frac{11.97mm}{16.47mm + 26.50mm} = 0.279$$

$$\alpha = 7.315$$

Effective length for one bolt in circular patterns

$$l_{eff,cp} = 2 \cdot \pi \cdot m_{ep} = 2 \cdot \pi \cdot 16.47mm = 103.51mm$$

Effective length for one bolt in non-circular patterns

$$l_{eff,nc} = \alpha \cdot m_{ep} = 7.315 \cdot 16.47mm = 120.51mm$$

Effective length for one bolt for 1 mode

$$l_{eff,1} = \min(l_{eff,cp}; l_{eff,nc}) = \min(103.51mm; 120.51mm) = 103.51mm$$

Effective length for one bolt for 2 mode

$$l_{eff,2} = l_{eff,nc} = 120.51mm$$

Model 1: Complete yielding of the end plate

$$M_{pl,1,Rd} = \frac{0.25 \cdot l_{eff,1} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 103.51mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 0.59kNm$$

Method 1



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$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 0.59 kNm}{16.47 mm} = 142.75 kN$$

Method 2 (alternative method)

Parameter for bearing zone

$$e_w = 0.25 \cdot d_w = 0.25 \cdot 20.00 mm = 5.00 mm$$

$$F_{T,1,Rd2} = \frac{(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{2 \cdot m_{ep} \cdot n - e_w \cdot (m_{ep} + n)} = \frac{(8 \cdot 20.59 mm - 2 \cdot 5.00 mm) \cdot 0.59 kNm}{2 \cdot 16.47 mm \cdot 20.59 mm - 5.00 mm \cdot (16.47 mm + 20.59 mm)} = 184.48 kN$$

$$F_{T,1,Rd} = \min(F_{T,1,Rd1}; F_{T,1,Rd2}) = \min(142.75 kN; 184.48 kN) = 142.75 kN$$

Model 2: Bolt failure with yielding of the end plate

$$M_{pl,2,Rd} = \frac{0.25 \cdot l_{eff,2} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 120.51 mm \cdot (8.00 mm)^2 \cdot 355.00 MPa}{1.000} = 0.68 kNm$$

$$F_{T,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \cdot \sum F_{t,Rd}}{m_{ep} + n} = \frac{2 \cdot 0.68 kNm + 20.59 mm \cdot 2 \cdot 33.41 kN}{16.47 mm + 20.59 mm} = 74.05 kN$$

Model 3: Bolt failure

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 33.41 kN = 66.82 kN$$

Component resistance

$$F_{t,ep,Rd(1)} = \min(F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}) = \min(142.75 kN; 74.05 kN; 66.82 kN) = 66.82 kN$$

Beam web in tension

Effective width of beam web in tension

$$b_{eff,t,wb} = l_{eff1(1)} = 103.51 mm$$

Component resistance

$$F_{t,wb,Rd(1)} = \frac{b_{eff,t,wb} \cdot t_{wb} \cdot f_{yb}}{\gamma_{M0}} = \frac{103.51 mm \cdot 5.00 mm \cdot 355.00 MPa}{1.000} = 183.73 kN$$

Resistance of the bolt row 1

$F_{t,Rd(1)M} = \min$		
·	$F_{t,ep,Rd(1)}$	= 66.82 kN



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·	$F_{t,wb,Rd(1)} =$	183.73kN
·	$F_{cfb,Rd(1)} =$	334.90kN
	$=$	66.82kN

BOLTS ROW 2

End plate in bending

Geometrical parameters

Bolt distance from outer edge

$$e_{ep} = 26.50mm$$

Bolt distance from beam web

$$m_{ep} = 0.5 \cdot (w - t_{wb}) - 0.8 \cdot \sqrt{2} \cdot a_w = 0.5 \cdot (47.00mm - 5.00mm) - 0.8 \cdot \sqrt{2} \cdot 4.00mm = 16.47mm$$

Distance e{min}

$$e_{min} = 26.50mm$$

Calculation parameter n

$$n = \min(e_{min}; 1.25 \cdot m_{ep}) = \min(26.50mm; 1.25 \cdot 16.47mm) = 20.59mm$$

Effective length for one bolt in circular patterns

$$l_{eff,cp} = 2 \cdot \pi \cdot m_{ep} = 2 \cdot \pi \cdot 16.47mm = 103.51mm$$

Effective length for one bolt in non-circular patterns

$$l_{eff,nc} = 4 \cdot m_{ep} + 1.25 \cdot e = 4 \cdot 16.47mm + 1.25 \cdot 26.50mm = 99.02mm$$

Effective length for one bolt for 1 mode

$$l_{eff,1} = \min(l_{eff,cp}; l_{eff,nc}) = \min(103.51mm; 99.02mm) = 99.02mm$$

Effective length for one bolt for 2 mode

$$l_{eff,2} = l_{eff,nc} = 99.02mm$$

Model 1: Complete yielding of the end plate

$$M_{pl,1,Rd} = \frac{0.25 \cdot l_{eff,1} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 99.02mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 0.56kNm$$

Method 1

$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 0.56kNm}{16.47mm} = 136.56kN$$



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Method 2 (alternative method)

Parameter for bearing zone

$$e_w = 0.25 \cdot d_w = 0.25 \cdot 20.00 \text{ mm} = 5.00 \text{ mm}$$

$$F_{T,1,Rd2} = \frac{(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{2 \cdot m_{ep} \cdot n - e_w \cdot (m_{ep} + n)} = \frac{(8 \cdot 20.59 \text{ mm} - 2 \cdot 5.00 \text{ mm}) \cdot 0.56 \text{ kNm}}{2 \cdot 16.47 \text{ mm} \cdot 20.59 \text{ mm} - 5.00 \text{ mm} \cdot (16.47 \text{ mm} + 20.59 \text{ mm})} = 176.48 \text{ kN}$$

$$F_{T,1,Rd} = \min(F_{T,1,Rd1}; F_{T,1,Rd2}) = \min(136.56 \text{ kN}; 176.48 \text{ kN}) = 136.56 \text{ kN}$$

Model 2: Bolt failure with yielding of the end plate

$$M_{pl,2,Rd} = \frac{0.25 \cdot l_{eff,2} \cdot t_p^2 \cdot f_{yp}}{\gamma_{M0}} = \frac{0.25 \cdot 99.02 \text{ mm} \cdot (8.00 \text{ mm})^2 \cdot 355.00 \text{ MPa}}{1.000} = 0.56 \text{ kNm}$$

$$F_{T,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \cdot \sum F_{t,Rd}}{m_{ep} + n} = \frac{2 \cdot 0.56 \text{ kNm} + 20.59 \text{ mm} \cdot 2 \cdot 33.41 \text{ kN}}{16.47 \text{ mm} + 20.59 \text{ mm}} = 67.47 \text{ kN}$$

Model 3: Bolt failure

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 33.41 \text{ kN} = 66.82 \text{ kN}$$

Component resistance

$$F_{t,ep,Rd(1)} = \min(F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}) = \min(136.56 \text{ kN}; 67.47 \text{ kN}; 66.82 \text{ kN}) = 66.82 \text{ kN}$$

Beam web in tension

Effective width of beam web in tension

$$b_{eff,t,wb} = l_{eff1(2)} = 99.02 \text{ mm}$$

Component resistance

$$F_{t,wb,Rd(2)} = \frac{b_{eff,t,wb} \cdot t_{wb} \cdot f_{yb}}{\gamma_{M0}} = \frac{99.02 \text{ mm} \cdot 5.00 \text{ mm} \cdot 355.00 \text{ MPa}}{1.000} = 175.77 \text{ kN}$$

End plate in bending

Row 1+2 considered as group

Circular patterns

Bolts row 1

$$l_{eff,cp(1,g)} = \pi \cdot m_{ep} + p = \pi \cdot 16.47 \text{ mm} + 47.00 \text{ mm} = 98.76 \text{ mm}$$

Bolts row 2

$$l_{eff,cp(2,g)} = \pi \cdot m_{ep} + p = \pi \cdot 16.47 \text{ mm} + 47.00 \text{ mm} = 98.76 \text{ mm}$$

$$\sum l_{eff(1+2)} = l_{eff,cp(1,g)} + l_{eff,cp(2,g)} = 98.76mm + 98.76mm = 197.51mm$$

Non-circular patterns

Bolts row 1

$$\begin{aligned} l_{eff,nc(1,g)} &= 0.5 \cdot p + \alpha \cdot m_{ep} - (2 \cdot m_{ep} + 0.625 \cdot e) = \\ &= 0.5 \cdot 47.00mm + 7.315 \cdot 16.47mm - (2 \cdot 16.47mm + 0.625 \cdot 26.50mm) = 94.49mm \end{aligned}$$

Bolts row 2

$$l_{eff,nc(2,g)} = 2 \cdot m_{ep} + 0.625 \cdot e + 0.5 \cdot p = 2 \cdot 16.47mm + 0.625 \cdot 26.50mm + 0.5 \cdot 47.00mm = 73.01mm$$

$$\sum l_{eff,nc(1+2)} = l_{eff,nc(1,g)} + l_{eff,nc(2,g)} = 94.49mm + 73.01mm = 167.51mm$$

Model 1: Complete yielding of the end plate

Effective length for one bolt for 1 mode

$$\sum l_{eff,1(1+2)} = \min(\sum l_{eff,cp(1+2)}; \sum l_{eff,nc(1+2)}) = \min(197.51mm; 167.51mm) = 167.51mm$$

$$M_{pl,1,Rd} = \frac{0.25 \cdot \sum l_{eff,1(1+2)} \cdot t_p^2 \cdot f_{yp}}{Y_{M0}} = \frac{0.25 \cdot 167.51mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 1.12kNm$$

Method 1

$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 1.12kNm}{16.47mm} = 272.39kN$$

$$F_{T,1,Rd1} = \frac{4 \cdot M_{pl,1,Rd}}{m_{ep}} = \frac{4 \cdot 1.12kNm}{16.47mm} = 272.39kN$$

Method 2 (alternative method)

$$F_{T,1,Rd2} = \frac{(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{2 \cdot m_{ep} \cdot n - e_w \cdot (m_{ep} + n)} = \frac{(8 \cdot 20.59mm - 2 \cdot 5.00mm) \cdot 1.12kNm}{2 \cdot 16.47mm \cdot 20.59mm - 5.00mm \cdot (16.47mm + 20.59mm)} = 352.01kN$$

$$F_{T,1,Rd2} = \frac{(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{2 \cdot m_{ep} \cdot n - e_w \cdot (m_{ep} + n)} = \frac{(8 \cdot 20.59mm - 2 \cdot 5.00mm) \cdot 1.12kNm}{2 \cdot 16.47mm \cdot 20.59mm - 5.00mm \cdot (16.47mm + 20.59mm)} = 352.01kN$$

$$F_{T,1,Rd} = \min(F_{T,1,Rd1}; F_{T,1,Rd2}) = \min(272.39kN; 352.01kN) = 272.39kN$$

Model 2: Bolt failure with yielding of the end plate

Effective length for one bolt for 2 mode

$$\sum l_{eff,2(1+2)} = \sum l_{eff,nc(1+2)} = 167.51mm$$

$$M_{pl,2,Rd} = \frac{0.25 \cdot \sum l_{eff,2(1+2)} \cdot t_p^2 \cdot f_{yp}}{Y_{M0}} = \frac{0.25 \cdot 167.51mm \cdot (8.00mm)^2 \cdot 355.00MPa}{1.000} = 1.12kNm$$



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$$F_{T,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \cdot \sum B_{p,Rd}}{m_{ep} + n} = \frac{2 \cdot 1.12kNm + 20.59mm \cdot 4 \cdot 33.41kN}{16.47mm + 20.59mm} = 134.77kN$$

Model 3: Bolt failure

$$F_{T,3,Rd} = \sum F_{t,Rd} = 4 \cdot 33.41kN = 133.63kN$$

Resistance of group

$$F_{t,ep(1+2)} = \min(F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd}) = \min(272.39kN; 134.77kN; 133.63kN) = 133.63kN$$

Beam web in tension

Effective width of beam web in tension

$$b_{eff,t,wb} = \sum l_{eff1(1+2)} = 197.51mm$$

Component resistance

$$F_{t,wb,Rd(1+2)} = \frac{b_{eff,t,wb} \cdot t_{wb} \cdot f_{yb}}{\gamma_{M0}} = \frac{197.51mm \cdot 5.00mm \cdot 355.00MPa}{1.000} = 350.58kN$$

Resistance of the bolt row 2

$F_{t,Rd(2)M} = \min$			
·	$F_{t,ep,Rd(2)} =$	66.82kN	
·	$F_{t,wb,Rd(2)} =$	175.77kN	
·	$F_{t,ep,Rd(1+2)} - F_{t,Rd(1)} =$	$133.63kN - 66.82kN =$	66.82kN
·	$F_{t,wb,Rd(1+2)} - F_{t,Rd(1)} =$	$350.58kN - 66.82kN =$	283.77kN
·	$F_{cfb,Rd} - F_{t,Rd(1)} =$	$334.90kN - 66.82kN =$	268.08kN
$= 66.82kN$			

Bending resistance

Real bending moment

$$M_0 = M_{b1,Ed} = 0.00kNm$$

The design moment resistance of the joint, assuming no axial force

$$M_{j,Rd} = F_{t,Rd(1)M} \cdot h_1 + F_{t,Rd(2)M} \cdot h_2 = 66.82kN \cdot 67.50mm + 66.82kN \cdot 20.50mm = 5.88kNm$$

$\frac{ M_0 }{M_{j,Rd}} \leq 1$	0.000 < 1.000	0.000	✓
---------------------------------	---------------	-------	---

Shear resistance



Shear force

$$V_0 = -N_{b1,Ed} \cdot \sin(\alpha_1) + V_{b1,Ed} \cdot \cos(\alpha_1) = -((-49.22kN)) \cdot \sin(0.00\text{Deg}) + 0.00kN \cdot \cos(0.00\text{Deg}) = 0.00kN$$

BOLTS ROW 1

Bolt bearing on the plate

Coefficient determined by bolt spacing

$$\alpha_b = \min(1.0; \frac{f_{ub}}{f_{up}}; \frac{e_1}{3 \cdot d_0}) = \min(1.0; \frac{800.00MPa}{490.00MPa}; \frac{24.50mm}{3 \cdot 11.00mm}) = 0.742$$

Coefficient determined by bolt spacing

$$k_1 = \min(2.5; 2.8 \cdot \frac{e_2}{d_0} - 1.7) = \min(2.5; 2.8 \cdot \frac{26.50mm}{11.00mm} - 1.7) = 2.500$$

Bearing resistance of a bolt

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p}{\gamma_{M2}} = \frac{2.500 \cdot 0.742 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 58.21kN$$

Resistance of the bolt row 1

$$V_{Rd(1)} = m_1 \cdot \min(F_{b,Rd}; F_{v,Rd}) = 2 \cdot \min(58.21kN; 30.16kN) = 60.32kN$$

BOLTS ROW 2

Bolt bearing on the plate

Coefficient determined by bolt spacing

$$\alpha_b = \min(1.0; \frac{f_{ub}}{f_{up}}; \frac{e_1}{3 \cdot d_0}) = \min(1.0; \frac{800.00MPa}{490.00MPa}; \frac{24.50mm}{3 \cdot 11.00mm}) = 0.742$$

Coefficient determined by bolt spacing

$$k_1 = \min(2.5; 2.8 \cdot \frac{e_2}{d_0} - 1.7) = \min(2.5; 2.8 \cdot \frac{26.50mm}{11.00mm} - 1.7) = 2.500$$

Bearing resistance of a bolt

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p}{\gamma_{M2}} = \frac{2.500 \cdot 0.742 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 58.21kN$$

Resistance of the bolt row 2

$$V_{Rd(2)} = m_2 \cdot \min(F_{b,Rd}; F_{v,Rd}) = 2 \cdot \min(58.21kN; 30.16kN) = 60.32kN$$



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$$V_{j,Rd} = V_{Rd(1)} + V_{Rd(2)} = 60.32kN + 60.32kN = 120.64kN$$

$\frac{ V_0 }{V_{j,Rd}} \leq 1$	$ 0.00kN < 120.64kN$	0.000	✓
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Butt welds connecting beam and front plate

Forces in welds

Axial force

$$N_0 = N_{b1,Ed} \cdot \cos(\alpha_1) + V_{b1,Ed} \cdot \sin(\alpha_1) = (-49.22kN) \cdot \cos(0.00Deg) + 0.00kN \cdot \sin(0.00Deg) = -49.22kN$$

Shear force

$$V_0 = -N_{b1,Ed} \cdot \sin(\alpha_1) + V_{b1,Ed} \cdot \cos(\alpha_1) = -((-49.22kN)) \cdot \sin(0.00Deg) + 0.00kN \cdot \cos(0.00Deg) = 0.00kN$$

Real bending moment

$$M_0 = M_{b1,Ed} = 0.00kNm$$

Geometric properties of welds

Beam

Area of horizontal welds on upper flange

$$A_{wfu} = [2 \cdot (b_{fb} - r_b) - t_{wb}] \cdot a_f = [2 \cdot (100.00mm - 12.00mm) - 5.00mm] \cdot 4.00mm = 6.84cm^2$$

Area of horizontal welds on lower flange

$$A_{wfl} = [2 \cdot (b_{fb} - r_b) - t_{wb}] \cdot a_f = [2 \cdot (100.00mm - 12.00mm) - 5.00mm] \cdot 4.00mm = 8.97cm^2$$

Area of vertical welds

$$A_{ww} = 2 \cdot \frac{h_b - 2 \cdot (t_{fb} + r_b)}{\cos(\alpha)} \cdot a_w = 2 \cdot \frac{96.00mm - 2 \cdot (8.00mm + 12.00mm)}{\cos(0.00Deg)} \cdot 4.00mm = 4.48cm^2$$

Area of all welds

$$A_w = A_{wfu} + A_{wfl} + A_{ww} = 6.84cm^2 + 8.97cm^2 + 4.48cm^2 = 20.29cm^2$$

Offset of welds centroid with respect to beam centroid

$$e_{0w} = -18.28mm$$

Moment of inertia welds

$$I_w = 237.83cm^4$$

The point where the stresses are checked

$$z_i = 66.28mm$$



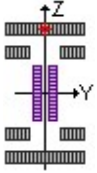
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Elastic modulus of welds	
$W_w = 35.88 \text{ cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.22 \text{ kN}}{20.29 \text{ cm}^2} = -24.26 \text{ MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{I_w} = \frac{0.00 \text{ kNm} \cdot 66.28 \text{ mm}}{237.83 \text{ cm}^4} = 0.00 \text{ MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -24.26 \text{ MPa} + (0.00 \text{ MPa}) = -24.26 \text{ MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \sigma = -24.26 \text{ MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -24.26 \text{ MPa} < 352.80 \text{ MPa}$	0.069	✓
---	---	-------	---

$\sqrt{\sigma_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$24.26 \text{ MPa} < 435.56 \text{ MPa}$	0.056	✓
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The point where the stresses are checked	$z_i = 46.28 \text{ mm}$
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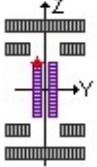
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Elastic modulus of welds	
$W_w = 51.39cm^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.22kN}{20.29cm^2} = -24.26MPa$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{I_w} = \frac{0.00kNm \cdot 46.28mm}{237.83cm^4} = 0.00MPa$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -24.26MPa + (0.00MPa) = -24.26MPa$	
Normal perpendicular stress	
$\sigma_{\perp} = \sigma = -24.26MPa$	
Parallel tangent stress	
$\tau_{ } = \frac{V_0}{A_{ww}} = \frac{0.00kN}{4.48cm^2} = 0.00MPa$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -24.26MPa < 352.80MPa$	0.069	✓
---	---------------------------	-------	---

$\sqrt{\sigma_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$24.26MPa < 435.56MPa$	0.056	✓
--	------------------------	-------	---

The point where the stresses are checked	$z_i = -9.72mm$
--	-----------------



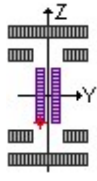
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Elastic modulus of welds	
$W_w = 244.66 \text{ cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.22 \text{ kN}}{20.29 \text{ cm}^2} = -24.26 \text{ MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{I_w} = \frac{0.00 \text{ kNm} \cdot (-9.72 \text{ mm})}{237.83 \text{ cm}^4} = 0.00 \text{ MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -24.26 \text{ MPa} + 0.00 \text{ MPa} = -24.26 \text{ MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \sigma = -24.26 \text{ MPa}$	
Parallel tangent stress	
$\tau_{\parallel} = \frac{V_0}{A_{ww}} = \frac{0.00 \text{ kN}}{4.48 \text{ cm}^2} = 0.00 \text{ MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -24.26 \text{ MPa} < 352.80 \text{ MPa}$	0.069	✓
---	---	-------	---

$\sqrt{\sigma_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$24.26 \text{ MPa} < 435.56 \text{ MPa}$	0.056	✓
--	--	-------	---

The point where the stresses are checked	$z_i = -29.72 \text{ mm}$
--	---------------------------



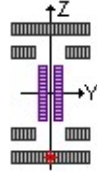
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Elastic modulus of welds	
$W_w = 80.02 \text{ cm}^3$	
Stress from axial force	
$\sigma_N = \frac{N_0}{A_w} = \frac{-49.22 \text{ kN}}{20.29 \text{ cm}^2} = -24.26 \text{ MPa}$	
Stress due to bending	
$\sigma_M = \frac{M_0 \cdot z_i}{I_w} = \frac{0.00 \text{ kNm} \cdot (-29.72 \text{ mm})}{237.83 \text{ cm}^4} = 0.00 \text{ MPa}$	
Maximum normal stress	
$\sigma = \sigma_N + \sigma_M = -24.26 \text{ MPa} + 0.00 \text{ MPa} = -24.26 \text{ MPa}$	
Normal perpendicular stress	
$\sigma_{\perp} = \sigma = -24.26 \text{ MPa}$	

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma_{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -24.26 \text{ MPa} < 352.80 \text{ MPa}$	0.069	✓
---	---	-------	---

$\sqrt{\sigma_{\perp}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$24.26 \text{ MPa} < 435.56 \text{ MPa}$	0.056	✓
--	--	-------	---

The rotational stiffness of a joint

The bolt elongation length

$$L_b = 2 \cdot t_p + 0.5 \cdot (m + k) + 2 \cdot t_{wa} = 2 \cdot 8.00 \text{ mm} + 0.5 \cdot (9.50 \text{ mm} + 6.40 \text{ mm}) + 2 \cdot 2.00 \text{ mm} = 27.95 \text{ mm}$$

$$k_{10} = \frac{3.2 \cdot A_s}{L_b} = \frac{3.2 \cdot 0.58 \text{ cm}^2}{27.95 \text{ mm}} = 3.32 \text{ mm}$$

Bolts row 1

End plate in bending

$$k_{5,1} = \frac{0.9 \cdot l_{eff} \cdot t_p^3}{m_{ep}^3} = \frac{0.9 \cdot 103.51 \text{ mm} \cdot (8.00 \text{ mm})^3}{(16.47 \text{ mm})^3} = 10.67 \text{ mm}$$



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$$k_{eff,1} = \frac{1}{\frac{1}{k_5} + \frac{1}{k_{10}}} = \frac{1}{\frac{1}{10.67mm} + \frac{1}{3.32mm}} = 2.53mm$$

Bolts row 2

End plate in bending

$$k_{5,2} = \frac{0.9 \cdot l_{eff} \cdot t_p^3}{m_{ep}^3} = \frac{0.9 \cdot 99.02mm \cdot (8.00mm)^3}{(16.47mm)^3} = 10.20mm$$

$$k_{eff,2} = \frac{1}{\frac{1}{k_5} + \frac{1}{k_{10}}} = \frac{1}{\frac{1}{10.20mm} + \frac{1}{3.32mm}} = 2.51mm$$

Lever arm of internal forces

$$z_{eq} = \frac{k_{eff,1} \cdot h_1^2 + k_{eff,2} \cdot h_2^2}{k_{eff,1} \cdot h_1 + k_{eff,2} \cdot h_2} = \frac{2.53mm \cdot (67.50mm)^2 + 2.51mm \cdot (20.50mm)^2}{2.53mm \cdot 67.50mm + 2.51mm \cdot 20.50mm} = 56.64mm$$

Equivalent stiffness coefficient

$$k_{eq} = \frac{k_{eff,1} \cdot h_1 + k_{eff,2} \cdot h_2}{z_{eq}} = \frac{2.53mm \cdot 67.50mm + 2.51mm \cdot 20.50mm}{56.64mm} = 3.92mm$$

The initial rotational stiffness of a joint

$$S_{j,ini} = \frac{\frac{E \cdot z_{eq}^2}{1}}{\frac{1}{k_{eq}}} = \frac{210000.00MPa \cdot (56.64mm)^2}{3.92mm} = 2643.84 \frac{kNm}{rad}$$

The rotational stiffness of a pinned joint

$$S_{j,pin} = \frac{0.5 \cdot E \cdot I_{yb}}{L_b} = \frac{0.5 \cdot 210000.00MPa \cdot 349.24cm^4}{2520.00mm} = 145.52 \frac{kNm}{rad}$$

The rotational stiffness of a rigid joint

$$S_{j,rig} = \frac{k_b \cdot E \cdot I_{yb}}{L_b} = \frac{8.000 \cdot 210000.00MPa \cdot 349.24cm^4}{2520.00mm} = 2328.24 \frac{kNm}{rad}$$

Stiffness ratio

$$\mu = 1.000$$

The rotational stiffness of a joint

$$S_j = \frac{S_{j,ini}}{\mu} = \frac{2643.84 \frac{kNm}{rad}}{1.000} = 2643.84 \frac{kNm}{rad}$$



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Classification of joint

Rigid



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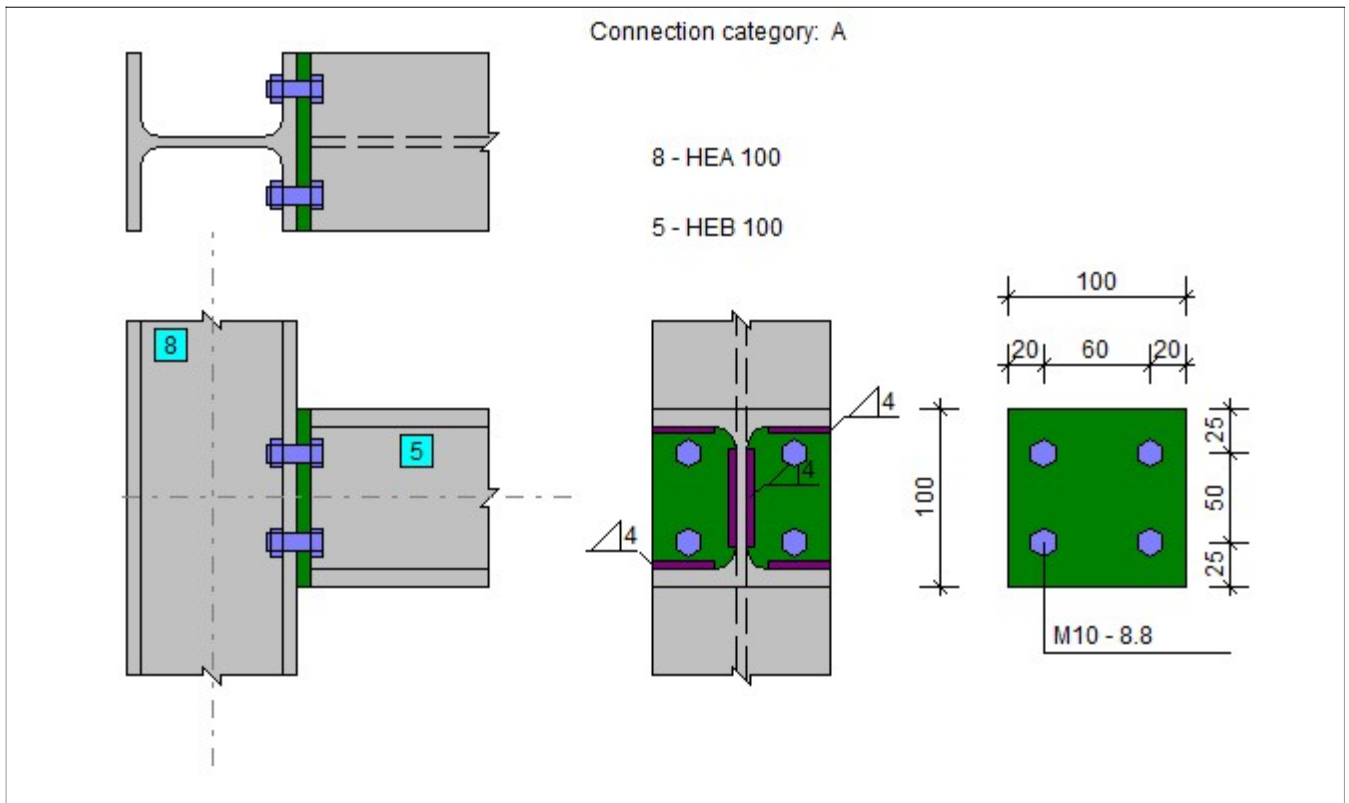
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	Beam-plate-column	EN 1991-1-8:2006	Ratio : 0.857	
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Structure node: 16

Structure members: 8, 5



Data

Column HEA 100					
	h_c	b_{fc}	t_{fc}	t_{wc}	R_c
	96.00mm	100.00mm	8.00mm	5.00mm	12.00mm
	A_c	J_{y0c}	J_{z0c}	y_{0c}	z_{0c}
	21.24cm ²	349.24cm ⁴	133.81cm ⁴	50.00mm	48.00mm
Material	Grade	f_y	f_u		
	S 355	355.00MPa	490.00MPa		

Beam HEB 100					
	h_b	b_{fb}	t_{fb}	t_{wb}	R_b
	100.00mm	100.00mm	10.00mm	6.00mm	12.00mm
	A_b	J_{y0b}	J_{z0b}	y_{0b}	z_{0b}
	26.04cm ²	449.56cm ⁴	167.27cm ⁴	50.00mm	50.00mm



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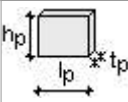
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Material	Grade	f_y	f_u
	S 355	355.00MPa	490.00MPa

Plate			
	l_p	h_p	t_p
	100.00mm	100.00mm	8.00mm
Material	Grade	f_y	f_u
	S 355	355.00MPa	490.00MPa

Bolts connecting front plate and column flange

Grade	8.8	
Yield point	$f_{yb} =$	640.00MPa
Resistance on tension	$f_{ub} =$	800.00MPa
Bolt diameter	$d =$	10.00mm
Bolt opening diameter	$d_0 =$	11.00mm
Area of bolt section	$A =$	0.79cm ²
Effective section area of a bolt	$A_s =$	0.58cm ²
Number of rows	$w =$	2.00
Number of cols	$k =$	2.00
Distance from horizontal edge	$e_1 =$	25.00mm
Distance from vertical edge	$e_2 =$	20.00mm
Vertical spacing	$p_1 =$	50.00mm
Horizontal spacing	$p_2 =$	60.00mm

Welds

Thickness of fillet welds connecting beam flanges and front plate	$a_f =$	4.00mm
Thickness of fillet welds connecting beam web and front plate	$a_w =$	4.00mm

Materials coeffs

Coefficient	$\gamma_{M0} =$	1.000
Coefficient	$\gamma_{M2} =$	1.250

Forces

Design loads [1.35*0.85*Gk] {1.5*Qk}



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Axial force	$N_{Ed} =$	$-6.55kN$	
Shear force	$V_{Ed} =$	$75.85kN$	

Results

Bolts connecting front plate and column flange

Tensile resistance of a bolt

$$F_{t,Rd} = \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{(0.900 \cdot 800.00MPa \cdot 0.58cm^2)}{1.250} = 33.41kN$$

Shear section area of bolt

$$A = 0.25 \cdot \pi \cdot d^2 = 0.25 \cdot 3.142 \cdot (10.00mm)^2 = 0.79cm^2$$

Shear resistance of bolt in one surface

$$F_{v,Rd} = \frac{\alpha_v \cdot m \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0.600 \cdot 1 \cdot 800.00MPa \cdot 0.79cm^2}{1.250} = 30.16kN$$

Bolt bearing on the column flange

	$e_{2_1} = 20.00mm$
$e_{2min} = \min(e_{2_1}) = \min(20.00mm) = 20.00mm$	

Direction X

Coefficient determined by bolt spacing

$$k_{1x} = \min\left(1.4 \cdot \frac{p_1}{d_0} - 1.7; 2.5\right) = \min\left(1.4 \cdot \frac{50.00mm}{11.00mm} - 1.7; 2.5\right) = 2.500$$

Coefficient determined by bolt spacing

$$\alpha_{bx} = \min\left(\frac{e_{2min}}{3 \cdot d_0}; \frac{p_2}{3 \cdot d_0} - 0.25; \frac{f_{ub}}{f_u}; 1.0\right) = \min\left(\frac{20.00mm}{3 \cdot 11.00mm}; \frac{60.00mm}{3 \cdot 11.00mm} - 0.25; \frac{800.00MPa}{490.00MPa}; 1.0\right) = 0.606$$

Bearing resistance of a bolt

$$F_{b,Rd1x} = \frac{k_{1x} \cdot \alpha_{bx} \cdot f_u \cdot d \cdot \sum t_i}{\gamma_{M2}} = \frac{2.500 \cdot 0.606 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 47.52kN$$



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Direction Z

Coefficient determined by bolt spacing

$$k_{1z} = \min\left(2.8 \cdot \frac{e_{2min}}{d_0} - 1.7; 1.4 \cdot \frac{p_2}{d_0} - 1.7; 2.5\right) = \min\left(2.8 \cdot \frac{20.00mm}{11.00mm} - 1.7; 1.4 \cdot \frac{60.00mm}{11.00mm} - 1.7; 2.5\right) = 2.500$$

Coefficient determined by bolt spacing

$$\alpha_{bz} = \min\left(\frac{p_1}{3 \cdot d_0} - 0.25; \frac{f_{ub}}{f_u}; 1.0\right) = \min\left(\frac{50.00mm}{3 \cdot 11.00mm} - 0.25; \frac{800.00MPa}{490.00MPa}; 1.0\right) = 1.000$$

Bearing resistance of a bolt

$$F_{b,Rd1z} = \frac{k_{1z} \cdot \alpha_{bz} \cdot f_u \cdot d \cdot \sum t_i}{\gamma_{M2}} = \frac{2.500 \cdot 1.000 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 78.40kN$$

Bolt bearing on the plate

	$e_{1_1} = 25.00mm$
	$e_{1_2} = 25.00mm$
	$e_{2_1} = 20.00mm$
$e_{1min} = \min(e_{1_1}; e_{1_2}) = \min(25.00mm; 25.00mm) = 25.00mm$	
$e_{2min} = \min(e_{2_1}) = \min(20.00mm) = 20.00mm$	

Direction X

Coefficient determined by bolt spacing

$$k_{1x} = \min\left(2.8 \cdot \frac{e_{1min}}{d_0} - 1.7; 1.4 \cdot \frac{p_1}{d_0} - 1.7; 2.5\right) = \min\left(2.8 \cdot \frac{25.00mm}{11.00mm} - 1.7; 1.4 \cdot \frac{50.00mm}{11.00mm} - 1.7; 2.5\right) = 2.500$$

Coefficient determined by bolt spacing

$$\alpha_{bx} = \min\left(\frac{e_{2min}}{3 \cdot d_0}; \frac{p_2}{3 \cdot d_0} - 0.25; \frac{f_{ub}}{f_u}; 1.0\right) = \min\left(\frac{20.00mm}{3 \cdot 11.00mm}; \frac{60.00mm}{3 \cdot 11.00mm} - 0.25; \frac{800.00MPa}{490.00MPa}; 1.0\right) = 0.606$$

Bearing resistance of a bolt

$$F_{b,Rd2x} = \frac{k_{1x} \cdot \alpha_{bx} \cdot f_u \cdot d \cdot \sum t_i}{\gamma_{M2}} = \frac{2.500 \cdot 0.606 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 47.52kN$$



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Direction Z

Coefficient determined by bolt spacing

$$k_{1z} = \min\left(2.8 \cdot \frac{e_{2min}}{d_0} - 1.7; 1.4 \cdot \frac{p_2}{d_0} - 1.7; 2.5\right) = \min\left(2.8 \cdot \frac{20.00mm}{11.00mm} - 1.7; 1.4 \cdot \frac{60.00mm}{11.00mm} - 1.7; 2.5\right) = 2.500$$

Coefficient determined by bolt spacing

$$\alpha_{bz} = \min\left(\frac{e_{1min}}{3 \cdot d_0}; \frac{p_1}{3 \cdot d_0} - 0.25; \frac{f_{ub}}{f_u}; 1.0\right) = \min\left(\frac{25.00mm}{3 \cdot 11.00mm}; \frac{50.00mm}{3 \cdot 11.00mm} - 0.25; \frac{800.00MPa}{490.00MPa}; 1.0\right) = 0.758$$

Bearing resistance of a bolt

$$F_{b,Rd2z} = \frac{k_{1z} \cdot \alpha_{bz} \cdot f_u \cdot d \cdot \sum t_i}{\gamma_{M2}} = \frac{2.500 \cdot 0.758 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 59.39kN$$

Ultimate limit state

Forces in bolts

Axial force

$$N_0 = N_{Ed} = -6.55kN$$

Shear force

$$V_0 = V_{Ed} = 75.85kN$$

Direction Z

Component force in a bolt due to influence of the shear force

$$F_{V,Ed} = \frac{V_0}{n_b} = \frac{75.85kN}{4} = 18.96kN$$

Total force in a bolt on the direction Z

$$F_{z,Ed} = |F_{V,Ed}| = 18.96kN$$

Effective design capacity of a bolt

$$F_{z,Rd} = \min(F_{b,Rd1z}; F_{b,Rd2z}) = \min(78.40kN; 59.39kN) = 59.39kN$$

$F_{z,Ed} \leq F_{z,Rd}$	$18.96kN < 59.39kN$	0.319	✓
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Resultant shear force in a bolt



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$$F_{Ed} = |F_{z,Ed}| = 18.96kN$$

$F_{Ed} \leq F_{v,Rd}$	$18.96kN < 30.16kN$	0.629	✓
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Block tearing

Forces in element

Shear force

$$V_0 = V_{Ed} = 75.85kN$$

Net area of the section in tension

$$A_{nt} = (w_1 - d_0) \cdot t = (60.00mm - 11.00mm) \cdot 8.00mm = 3.92cm^2$$

Net area of the section in shear

$$A_{nv} = 2 \cdot [e_1 + (n_v - 1) \cdot p_1 - (n_v - 0.5) \cdot d_0] \cdot t = 2 \cdot [25.00mm + (2 - 1) \cdot 50.00mm - (2 - 0.5) \cdot 11.00mm] \cdot 8.00mm = 9.36cm^2$$

Block tearing resistance

$$V_{eff,1,Rd} = \frac{f_u \cdot A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} \cdot \frac{f_y \cdot A_{nv}}{\gamma_{M0}} = \frac{490.00MPa \cdot 3.92cm^2}{1.250} + \frac{1}{\sqrt{3}} \cdot \frac{355.00MPa \cdot 9.36cm^2}{1.000} = 345.51kN$$

$ V_0 \leq V_{eff,1,Rd}$	$ 75.85kN < 345.51kN$	0.220	✓
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Net area of the section in tension

$$A_{nt} = 2 \cdot (e_2 - 0.5 \cdot d_0) \cdot t = 2 \cdot (20.00mm - 11.00mm) \cdot 8.00mm = 2.32cm^2$$

Net area of the section in shear

$$A_{nv} = 2 \cdot [e_1 + (n_v - 1) \cdot p_1 - (n_v - 0.5) \cdot d_0] \cdot t = 2 \cdot [25.00mm + (2 - 1) \cdot 50.00mm - (2 - 0.5) \cdot 11.00mm] \cdot 8.00mm = 9.36cm^2$$

Block tearing resistance

$$V_{eff,1,Rd} = \frac{f_u \cdot A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} \cdot \frac{f_y \cdot A_{nv}}{\gamma_{M0}} = \frac{490.00MPa \cdot 2.32cm^2}{1.250} + \frac{1}{\sqrt{3}} \cdot \frac{355.00MPa \cdot 9.36cm^2}{1.000} = 282.79kN$$

$ V_0 \leq V_{eff,1,Rd}$	$ 75.85kN < 282.79kN$	0.268	✓
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Beam

Shear resistance

$$V_{pw,Rd} = \frac{0.9 \cdot A_{vc} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{0.9 \cdot 4.80cm^2 \cdot 355.00MPa}{\sqrt{3} \cdot 1.000} = 88.54kN$$



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$ V_{Ed} \leq V_{wp,Rd}$	$ 75.85kN < 88.54kN$	0.857	✓
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Fillet welds connecting beam and front plate

Forces in welds

Axial force

$$N_0 = N_{Ed} = -6.55kN$$

Shear force

$$V_0 = V_{Ed} = 75.85kN$$

Geometric properties of welds

Area of horizontal welds on upper flange

$$A_{wfu} = (l_p - t_{wb} - 2 \cdot r_b) \cdot a_f = (100.00mm - 6.00mm - 2 \cdot 12.00mm) \cdot 4.00mm = 2.80cm^2$$

Area of horizontal welds on lower flange

$$A_{wfl} = (l_p - t_{wb} - 2 \cdot r_b) \cdot a_f = (100.00mm - 6.00mm - 2 \cdot 12.00mm) \cdot 4.00mm = 2.80cm^2$$

Area of vertical welds

$$A_{ww} = 4.48cm^2$$

Area of all welds

$$A_w = A_{wfu} + A_{wfl} + A_{ww} = 2.80cm^2 + 2.80cm^2 + 4.48cm^2 = 10.08cm^2$$

Stress from axial force

$$\sigma = \frac{N_0}{A_w} = \frac{-6.55kN}{10.08cm^2} = -6.50MPa$$

Normal perpendicular stress

$$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-6.50MPa}{\sqrt{2}} = -4.59MPa$$

Perpendicular tangent stress

$$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-6.50MPa}{\sqrt{2}} = -4.59MPa$$

Parallel tangent stress

$$\tau_{||} = \frac{V_0}{A_{ww}} = \frac{75.85kN}{4.48cm^2} = 169.32MPa$$

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma _{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -4.59MPa < 352.80MPa$	0.013	✓
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$\sqrt{\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$293.41 MPa < 435.56 MPa$	0.674	✓
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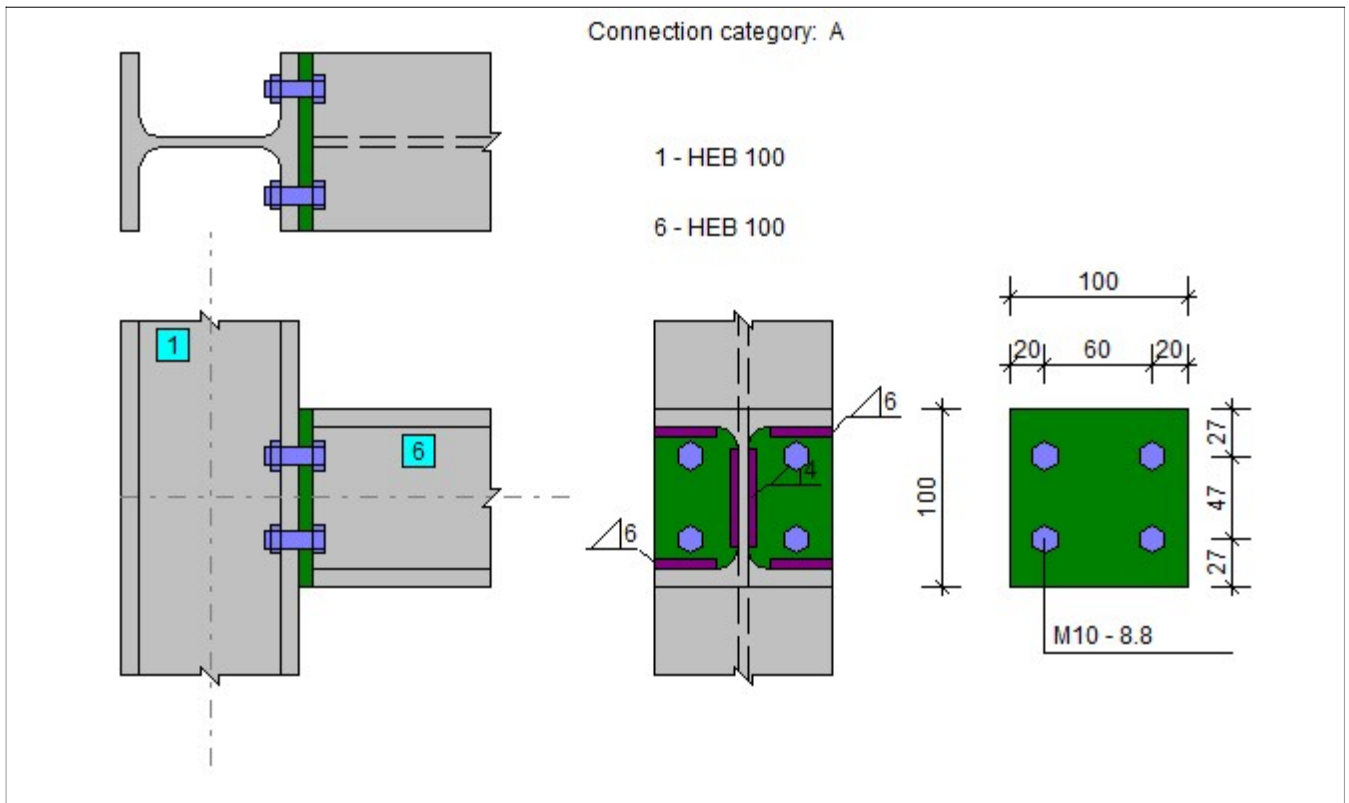
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	Beam-plate-column	EN 1991-1-8:2006	Ratio : 0.855	
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Structure node: 1

Structure members: 6, 1



Data

Column HEB 100					
	h_c	b_{fc}	t_{fc}	t_{wc}	R_c
	100.00mm	100.00mm	10.00mm	6.00mm	12.00mm
	A_c	J_{y0c}	J_{z0c}	y_{0c}	z_{0c}
	26.04cm ²	449.56cm ⁴	167.27cm ⁴	50.00mm	50.00mm
Material	Grade	f_y	f_u		
	S 355	355.00MPa	490.00MPa		

Beam HEB 100					
	h_b	b_{fb}	t_{fb}	t_{wb}	R_b
	100.00mm	100.00mm	10.00mm	6.00mm	12.00mm
	A_b	J_{y0b}	J_{z0b}	y_{0b}	z_{0b}
	26.04cm ²	449.56cm ⁴	167.27cm ⁴	50.00mm	50.00mm



Magnētiskās rezonanses iekārtas uzstādīšana Slimnīcas ielā 25, Lie

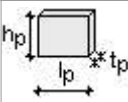
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Material	Grade	f_y	f_u
	S 355	355.00MPa	490.00MPa

Plate			
	l_p	h_p	t_p
	100.00mm	100.00mm	8.00mm
Material	Grade	f_y	f_u
	S 355	355.00MPa	490.00MPa

Bolts connecting front plate and column flange

Grade	8.8	
Yield point	$f_{yb} =$	640.00MPa
Resistance on tension	$f_{ub} =$	800.00MPa
Bolt diameter	$d =$	10.00mm
Bolt opening diameter	$d_0 =$	11.00mm
Area of bolt section	$A =$	0.79cm ²
Effective section area of a bolt	$A_s =$	0.58cm ²
Number of rows	$w =$	2.00
Number of cols	$k =$	2.00
Distance from horizontal edge	$e_1 =$	26.50mm
Distance from vertical edge	$e_2 =$	20.00mm
Vertical spacing	$p_1 =$	47.00mm
Horizontal spacing	$p_2 =$	60.00mm

Welds

Thickness of fillet welds connecting beam flanges and front plate	$a_f =$	6.00mm
Thickness of fillet welds connecting beam web and front plate	$a_w =$	4.00mm

Materials coeffs

Coefficient	$\gamma_{M0} =$	1.000
Coefficient	$\gamma_{M2} =$	1.250

Forces

Design loads [1.35*0.85*Gk] {1.5*Qk}



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Axial force	$N_{Ed} =$	$-6.55kN$	
Shear force	$V_{Ed} =$	$-75.72kN$	

Results

Bolts connecting front plate and column flange

Tensile resistance of a bolt

$$F_{t,Rd} = \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{(0.900 \cdot 800.00MPa \cdot 0.58cm^2)}{1.250} = 33.41kN$$

Shear section area of bolt

$$A = 0.25 \cdot \pi \cdot d^2 = 0.25 \cdot 3.142 \cdot (10.00mm)^2 = 0.79cm^2$$

Shear resistance of bolt in one surface

$$F_{v,Rd} = \frac{\alpha_v \cdot m \cdot f_{ub} \cdot A}{\gamma_{M2}} = \frac{0.600 \cdot 1 \cdot 800.00MPa \cdot 0.79cm^2}{1.250} = 30.16kN$$

Bolt bearing on the column flange

	$e_{2_1} = 20.00mm$
$e_{2min} = \min(e_{2_1}) = \min(20.00mm) = 20.00mm$	

Direction X

Coefficient determined by bolt spacing

$$k_{1x} = \min\left(1.4 \cdot \frac{p_1}{d_0} - 1.7; 2.5\right) = \min\left(1.4 \cdot \frac{47.00mm}{11.00mm} - 1.7; 2.5\right) = 2.500$$

Coefficient determined by bolt spacing

$$\alpha_{bx} = \min\left(\frac{e_{2min}}{3 \cdot d_0}; \frac{p_2}{3 \cdot d_0} - 0.25; \frac{f_{ub}}{f_u}; 1.0\right) = \min\left(\frac{20.00mm}{3 \cdot 11.00mm}; \frac{60.00mm}{3 \cdot 11.00mm} - 0.25; \frac{800.00MPa}{490.00MPa}; 1.0\right) = 0.606$$

Bearing resistance of a bolt

$$F_{b,Rd1x} = \frac{k_{1x} \cdot \alpha_{bx} \cdot f_u \cdot d \cdot \sum t_i}{\gamma_{M2}} = \frac{2.500 \cdot 0.606 \cdot 490.00MPa \cdot 10.00mm \cdot 10.00mm}{1.250} = 59.39kN$$



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Direction Z

Coefficient determined by bolt spacing

$$k_{1z} = \min\left(2.8 \cdot \frac{e_{2min}}{d_0} - 1.7; 1.4 \cdot \frac{p_2}{d_0} - 1.7; 2.5\right) = \min\left(2.8 \cdot \frac{20.00mm}{11.00mm} - 1.7; 1.4 \cdot \frac{60.00mm}{11.00mm} - 1.7; 2.5\right) = 2.500$$

Coefficient determined by bolt spacing

$$\alpha_{bz} = \min\left(\frac{p_1}{3 \cdot d_0} - 0.25; \frac{f_{ub}}{f_u}; 1.0\right) = \min\left(\frac{47.00mm}{3 \cdot 11.00mm} - 0.25; \frac{800.00MPa}{490.00MPa}; 1.0\right) = 1.000$$

Bearing resistance of a bolt

$$F_{b,Rd1z} = \frac{k_{1z} \cdot \alpha_{bz} \cdot f_u \cdot d \cdot \sum t_i}{\gamma_{M2}} = \frac{2.500 \cdot 1.000 \cdot 490.00MPa \cdot 10.00mm \cdot 10.00mm}{1.250} = 98.00kN$$

Bolt bearing on the plate

	$e_{1_1} = 26.50mm$
	$e_{1_2} = 26.50mm$
	$e_{2_1} = 20.00mm$
$e_{1min} = \min(e_{1_1}; e_{1_2}) = \min(26.50mm; 26.50mm) = 26.50mm$	
$e_{2min} = \min(e_{2_1}) = \min(20.00mm) = 20.00mm$	

Direction X

Coefficient determined by bolt spacing

$$k_{1x} = \min\left(2.8 \cdot \frac{e_{1min}}{d_0} - 1.7; 1.4 \cdot \frac{p_1}{d_0} - 1.7; 2.5\right) = \min\left(2.8 \cdot \frac{26.50mm}{11.00mm} - 1.7; 1.4 \cdot \frac{47.00mm}{11.00mm} - 1.7; 2.5\right) = 2.500$$

Coefficient determined by bolt spacing

$$\alpha_{bx} = \min\left(\frac{e_{2min}}{3 \cdot d_0}; \frac{p_2}{3 \cdot d_0} - 0.25; \frac{f_{ub}}{f_u}; 1.0\right) = \min\left(\frac{20.00mm}{3 \cdot 11.00mm}; \frac{60.00mm}{3 \cdot 11.00mm} - 0.25; \frac{800.00MPa}{490.00MPa}; 1.0\right) = 0.606$$

Bearing resistance of a bolt

$$F_{b,Rd2x} = \frac{k_{1x} \cdot \alpha_{bx} \cdot f_u \cdot d \cdot \sum t_i}{\gamma_{M2}} = \frac{2.500 \cdot 0.606 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 47.52kN$$



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Direction Z

Coefficient determined by bolt spacing

$$k_{1z} = \min\left(2.8 \cdot \frac{e_{2min}}{d_0} - 1.7; 1.4 \cdot \frac{p_2}{d_0} - 1.7; 2.5\right) = \min\left(2.8 \cdot \frac{20.00mm}{11.00mm} - 1.7; 1.4 \cdot \frac{60.00mm}{11.00mm} - 1.7; 2.5\right) = 2.500$$

Coefficient determined by bolt spacing

$$\alpha_{bz} = \min\left(\frac{e_{1min}}{3 \cdot d_0}; \frac{p_1}{3 \cdot d_0} - 0.25; \frac{f_{ub}}{f_u}; 1.0\right) = \min\left(\frac{26.50mm}{3 \cdot 11.00mm}; \frac{47.00mm}{3 \cdot 11.00mm} - 0.25; \frac{800.00MPa}{490.00MPa}; 1.0\right) = 0.803$$

Bearing resistance of a bolt

$$F_{b,Rd2z} = \frac{k_{1z} \cdot \alpha_{bz} \cdot f_u \cdot d \cdot \sum t_i}{\gamma_{M2}} = \frac{2.500 \cdot 0.803 \cdot 490.00MPa \cdot 10.00mm \cdot 8.00mm}{1.250} = 62.96kN$$

Ultimate limit state

Forces in bolts

Axial force

$$N_0 = N_{Ed} = -6.55kN$$

Shear force

$$V_0 = V_{Ed} = -75.72kN$$

Direction Z

Component force in a bolt due to influence of the shear force

$$F_{V,Ed} = \frac{V_0}{n_b} = \frac{-75.72kN}{4} = -18.93kN$$

Total force in a bolt on the direction Z

$$F_{z,Ed} = |F_{V,Ed}| = 18.93kN$$

Effective design capacity of a bolt

$$F_{z,Rd} = \min(F_{b,Rd1z}; F_{b,Rd2z}) = \min(98.00kN; 62.96kN) = 62.96kN$$

$F_{z,Ed} \leq F_{z,Rd}$	$18.93kN < 62.96kN$	0.301	✓
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Resultant shear force in a bolt



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$$F_{Ed} = |F_{z,Ed}| = 18.93kN$$

$F_{Ed} \leq F_{v,Rd}$	$18.93kN < 30.16kN$	0.628	✓
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Block tearing

Forces in element

Shear force

$$V_0 = V_{Ed} = -75.72kN$$

Net area of the section in tension

$$A_{nt} = (w_1 - d_0) \cdot t = (60.00mm - 11.00mm) \cdot 8.00mm = 3.92cm^2$$

Net area of the section in shear

$$A_{nv} = 2 \cdot [e_1 + (n_v - 1) \cdot p_1 - (n_v - 0.5) \cdot d_0] \cdot t = 2 \cdot [26.50mm + (2 - 1) \cdot 47.00mm - (2 - 0.5) \cdot 11.00mm] \cdot 8.00mm = 9.12cm^2$$

Block tearing resistance

$$V_{eff,1,Rd} = \frac{f_u \cdot A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} \cdot \frac{f_y \cdot A_{nv}}{\gamma_{M0}} = \frac{490.00MPa \cdot 3.92cm^2}{1.250} + \frac{1}{\sqrt{3}} \cdot \frac{355.00MPa \cdot 9.12cm^2}{1.000} = 340.59kN$$

$ V_0 \leq V_{eff,1,Rd}$	$ -75.72kN < 340.59kN$	0.222	✓
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Net area of the section in tension

$$A_{nt} = 2 \cdot (e_2 - 0.5 \cdot d_0) \cdot t = 2 \cdot (20.00mm - 11.00mm) \cdot 8.00mm = 2.32cm^2$$

Net area of the section in shear

$$A_{nv} = 2 \cdot [e_1 + (n_v - 1) \cdot p_1 - (n_v - 0.5) \cdot d_0] \cdot t = 2 \cdot [26.50mm + (2 - 1) \cdot 47.00mm - (2 - 0.5) \cdot 11.00mm] \cdot 8.00mm = 9.12cm^2$$

Block tearing resistance

$$V_{eff,1,Rd} = \frac{f_u \cdot A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}} \cdot \frac{f_y \cdot A_{nv}}{\gamma_{M0}} = \frac{490.00MPa \cdot 2.32cm^2}{1.250} + \frac{1}{\sqrt{3}} \cdot \frac{355.00MPa \cdot 9.12cm^2}{1.000} = 277.87kN$$

$ V_0 \leq V_{eff,1,Rd}$	$ -75.72kN < 277.87kN$	0.272	✓
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Beam

Shear resistance

$$V_{pw,Rd} = \frac{0.9 \cdot A_{vc} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{0.9 \cdot 4.80cm^2 \cdot 355.00MPa}{\sqrt{3} \cdot 1.000} = 88.54kN$$



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$ V_{Ed} \leq V_{wp,Rd}$	$ -75.72kN < 88.54kN$	0.855	✓
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Fillet welds connecting beam and front plate

Forces in welds

Axial force

$$N_0 = N_{Ed} = -6.55kN$$

Shear force

$$V_0 = V_{Ed} = -75.72kN$$

Geometric properties of welds

Area of horizontal welds on upper flange

$$A_{wfu} = (l_p - t_{wb} - 2 \cdot r_b) \cdot a_f = (100.00mm - 6.00mm - 2 \cdot 12.00mm) \cdot 6.00mm = 4.20cm^2$$

Area of horizontal welds on lower flange

$$A_{wfl} = (l_p - t_{wb} - 2 \cdot r_b) \cdot a_f = (100.00mm - 6.00mm - 2 \cdot 12.00mm) \cdot 6.00mm = 4.20cm^2$$

Area of vertical welds

$$A_{ww} = 4.48cm^2$$

Area of all welds

$$A_w = A_{wfu} + A_{wfl} + A_{ww} = 4.20cm^2 + 4.20cm^2 + 4.48cm^2 = 12.88cm^2$$

Stress from axial force

$$\sigma = \frac{N_0}{A_w} = \frac{-6.55kN}{12.88cm^2} = -5.08MPa$$

Normal perpendicular stress

$$\sigma_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-5.08MPa}{\sqrt{2}} = -3.59MPa$$

Perpendicular tangent stress

$$\tau_{\perp} = \frac{\sigma}{\sqrt{2}} = \frac{-5.08MPa}{\sqrt{2}} = -3.59MPa$$

Parallel tangent stress

$$\tau_{||} = \frac{V_0}{A_{ww}} = \frac{-75.72kN}{4.48cm^2} = -169.01MPa$$

Welds resistance coefficient

$$\beta_w = 0.900$$

$ \sigma _{\perp} \leq \frac{0.9 \cdot f_u}{\gamma_{M2}}$	$ -3.59MPa < 352.80MPa$	0.010	✓
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$\sqrt{\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$	$292.82 MPa < 435.56 MPa$	0.672	✓
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STEEL MEMBER DESIGN

 Design member **6**

 Nodes: **1-16**

 Code: **Eurocode [LV]**

 Material: **S 355**

 Cross-section: **HEB 100**

 Load case: **Linear,(Auto) Critical**

 Coefficient for seismic forces: **1.0**

1. Axial force-Bending-Shear

EN 1993-1-1: 6.2.1, 6.2.8, 6.2.9

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 1.000 \cdot L = 1.000 \cdot 60 = 60$ cm

$$N_{Ed} = -6.5 \text{ kN} \quad V_{y,Ed} = 0.063 \text{ kN} \quad V_{z,Ed} = -76 \text{ kN} \quad M_{y,Ed} = -3534 \text{ kNm} = -35.341 \text{ kNm} \quad M_{z,Ed} = -3.8 \text{ kNm} = -0.038 \text{ kNm} \quad M_{x,Ed} = -9.3 \text{ kNm} = -0.093 \text{ kNm}$$

$$\eta_{NMV_{pl}} = \max(\eta_N; \eta_{M_{y,pl}}; \eta_{M_{z,pl}}; \eta_{V_z}; \eta_{V_y}) = \max(0.7; 95.5; 0.2; 41.4; 0) = 95.5 \% \quad \text{passed}$$

2. Axial Force-Bending-Flexural Buckling

EN 1993-1-1: 6.3.3, Annex B: Method 2

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 1.000 \cdot L = 1.000 \cdot 60 = 60$ cm

$$C_{my} = \max(0.2 + 0.8 \cdot \alpha_{my}, 0.4) = \max(0.2 + 0.8 \cdot 0.357, 0.4) = 0.486 \geq 0.4 \quad \text{Table B.3}$$

$$C_{mz} = \max(0.2 + 0.8 \cdot \alpha_{mz}, 0.4) = \max(0.2 + 0.8 \cdot 0.499, 0.4) = 0.599 \geq 0.4 \quad \text{Table B.3}$$

$$f_{yy} = \min(\lambda_y^* - 0.2; 0.8) = \min(0.19 - 0.2; 0.8) = 0$$

$$f_{zz} = \min(2 \cdot \lambda_z^* - 0.6; 1.4) = \min(2 \cdot 0.31 - 0.6; 1.4) = 0.02$$

$$k_{yy} = C_{my} \cdot \left(1 + f_{yy} \cdot \frac{|N_{Ed}|}{\chi_y \cdot N_{pl,Rd}} \right) = 0.486 \cdot \left(1 + 0 \cdot \frac{|(-6.5)|}{\frac{1.00 \cdot 924}{1}} \right) = 0.486$$

$$k_{zy} = 0.6 \cdot k_{yy} = 0.6 \cdot 0.486 = 0.291 \quad \text{Table Annex B.1}$$

$$k_{yz} = 0.6 \cdot k_{zz} = 0.6 \cdot 0.599 = 0.36$$

$$k_{zz} = C_{mz} \cdot \left(1 + f_{zz} \cdot \frac{|N_{Ed}|}{\chi_z \cdot N_{pl,Rd}} \right) = 0.599 \cdot \left(1 + 0.02 \cdot \frac{|(-6.5)|}{\frac{0.94 \cdot 924}{1}} \right) = 0.599 \quad \text{Table Annex B.1}$$

$$\chi_y = \min\left(\frac{1}{\phi_y + \sqrt{\phi_y^2 - \lambda_y^{*2}}}; 1\right) = 1.00 \quad (6.49)$$

$$\chi_z = \min\left(\frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_z^{*2}}}; 1\right) = 0.94 \quad (6.49)$$

$$\eta_{NMBuckl_1} = \frac{|N_{Ed}|}{\chi_y \cdot A \cdot f_y} + k_{yy} \cdot \frac{|M_{y,Ed}|}{W_{pl,y} \cdot f_y} + k_{yz} \cdot \frac{|M_{z,Ed}|}{W_{pl,z} \cdot f_y} = \frac{|(-6.5)|}{\frac{1.00 \cdot 26 \cdot 36}{1}} + 0.486 \cdot \frac{|(-3534)|}{\frac{104 \cdot 36}{1}} + 0.36 \cdot \frac{|(-3.8)|}{\frac{51 \cdot 36}{1}} = 47.2 \% \quad (6.61)$$



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$$\eta_{NMBuckl_2} = \frac{|N_{Ed}|}{\frac{\chi_z \cdot A \cdot f_y}{\gamma_{M1}}} + k_{zy} \cdot \frac{|M_{y,Ed}|}{\frac{W_{ply} \cdot f_y}{\gamma_{M1}}} + k_{zz} \cdot \frac{|M_{z,Ed}|}{\frac{W_{pl,z} \cdot f_y}{\gamma_{M1}}} = \frac{|(-6.5)|}{\frac{0.94 \cdot 26 \cdot 36}{1}} + 0.291 \cdot \frac{|(-3534)|}{\frac{104 \cdot 36}{1}} + 0.599 \cdot \frac{|(-3.8)|}{\frac{51 \cdot 36}{1}} = 28.7 \% \quad (6.62)$$

$$\eta_{NMBuckl} = 47.2 \% \quad \text{passed}$$

3. Axial force-Bending-Lateral torsional buckling

EN 1993-1-1: 6.3.3, Annex B: Method 2

Disabled

4. Cross-section resistance to shear (y):

EN 1993-1-1: 6.2.6, 6.2.7

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 0.000 \cdot L = 0.000 \cdot 60 = 0 \text{ cm}$

$$A_{Vy} = 2 \cdot b \cdot t_f = 20 \text{ cm}^2$$

$$V_{pl,Rd,y} = \frac{A_{Vy} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{20 \cdot 36}{\sqrt{3} \cdot 1} = 410 \text{ kN} \quad (6.18)$$

$$M_{x,Ed} = -9.3 \text{ kNcm}$$

$$V_{pl,T,Rd,y} = \left(\sqrt{1 - \frac{\tau_{T,xy,Ed}}{1.25 \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}}}} \right) \cdot V_{pl,Rd,y} = \left(\sqrt{1 - \frac{0.99}{1.25 \cdot \frac{36}{\sqrt{3} \cdot 1}}} \right) \cdot 410 = 402 \text{ kN} \quad (6.26)$$

$$\eta_{Vy} = \frac{|V_{y,Ed}|}{V_{pl,T,Rd,y}} = \frac{|0.063|}{402} = 0 \% \quad (6.17) \quad \text{passed}$$

5. Shear web buckling resistance:

EN 1993-1-5: 5.1, 5.2, 5.3, 5.5, Annex A: A.3

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 0.000 \cdot L = 0.000 \cdot 60 = 0 \text{ cm}$

$$a_{max} = 0.6$$

$$\eta_w = 1.2 \quad 5.2 (2) \text{ Note 2}$$

$$\varepsilon = 0.81$$

$$h_w = h - 2 \cdot t_f = 10 - 2 \cdot 1 = 8 \text{ cm}$$

$$\frac{h_w}{t_w} \leq \frac{72 \cdot \varepsilon}{\eta_w} \rightarrow V_{b,Rd} = V_{pl,Rd,z} = 185 \text{ kN} \quad (5.1 (2))$$

$$\eta_{Vw} = \frac{|V_{z,Ed}|}{V_{b,Rd}} = \frac{|(-76)|}{185} = 41.0 \% \quad (5.10) \quad \text{passed}$$

6. Web shear-Bending-Axial force

EN 1993-1-1: 6.2.9; EN 1993-1-5: 7.1

 Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**

 Section class: **1** (Plastic design)

 Critical section: $x = 1.000 \cdot L = 1.000 \cdot 60 = 60 \text{ cm}$

$$M_{f,Rd} = b \cdot t_f \cdot f_y \cdot (h - t_f) = 10 \cdot 1 \cdot 36 \cdot (10 - 1) = 3195 \text{ kNcm} = 31.950 \text{ kNm}$$



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$$\eta_{V_w^{MN}} = \frac{|M_{y,Ed}|}{M_{pl,Rd,y}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd,y}}\right) \left(2 \cdot \frac{V_{z,Ed}}{V_{bw,Rd}} - 1\right)^2 = \frac{|(-3534)|}{3700} + \left(1 - \frac{3195}{3700}\right) \left(2 \cdot \frac{(-76)}{118} - 1\right)^2 = 96.6 \% \quad (7.1) \quad \text{passed}$$

7. SLS (Serviceability Limit State)

EN 1993-1-1: 7., EN 1990: 3.4, A1.4.

Critical combination: **[Gk] {Qk}**Section class: **1** (Plastic design)Critical section: $x = 0.600 \cdot L = 0.600 \cdot 60 = 36$ cm

$$e_z = \left| e_{z,i} - e_{i,z} \cdot \left(1 - \frac{x}{L}\right) - e_{j,z} \cdot \frac{x}{L} + u_z \right| = \left| (-0.23) - (-0.027) \cdot \left(1 - \frac{36}{60}\right) - (-0.29) \cdot \frac{36}{60} + 0 \right| = 0.044 \text{ cm}$$

$$e_{z,Limit} = \frac{L}{300.0} = \frac{60}{300.0} = 0.2 \text{ cm}$$

$$\eta_e = \frac{e_z}{e_{z,Limit}} = \frac{0.044}{0.2} = 21.8 \%$$

$$\eta_{SLS} = \max(\eta_e) = \max(21.8) = 21.8 \% \quad \text{passed}$$

Partial results

8. Cross-section resistance to axial force:

EN 1993-1-1: 6.2.4

Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**Section class: **1** (Plastic design)Critical section: $x = 0.000 \cdot L = 0.000 \cdot 60 = 0$ cm

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{26 \cdot 36}{1} = 924 \text{ kN} \quad (6.10)$$

$$\eta_N = \frac{|N_{Ed}|}{N_{pl,Rd}} = \frac{|(-6.5)|}{924} = 0.7 \% \quad (6.9) \quad \text{passed}$$

9. Cross-section resistance to bending (yy):

EN 1993-1-1: 6.2.5

Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**Section class: **1** (Plastic design)Critical section: $x = 1.000 \cdot L = 1.000 \cdot 60 = 60$ cm

$$M_{pl,Rd,y} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{104 \cdot 36}{1} = 3700 \text{ kNcm} = 36.997 \text{ kNm} \quad (6.13)$$

$$\eta_{M_{y,pl}} = \frac{|M_{y,Ed}|}{M_{pl,Rd,y}} = \frac{|(-3534)|}{3700} = 95.5 \% \quad (6.12) \quad \text{passed}$$

10. Cross-section resistance to bending (zz):

EN 1993-1-1: 6.2.5

Critical combination: **[1.35*0.85*Gk] {1.5*Qk}**Section class: **1** (Plastic design)Critical section: $x = 1.000 \cdot L = 1.000 \cdot 60 = 60$ cm



$$M_{pl,Rd,z} = \frac{W_{pl,z} \cdot f_y}{\gamma_{M0}} = \frac{51 \cdot 36}{1} = 1826 \text{ kNcm} = 18.255 \text{ kNm} \quad (6.13)$$

$$\eta_{M_{z,pl}} = \frac{|M_{z,Ed}|}{M_{pl,Rd,z}} = \frac{|(-3.8)|}{1826} = 0.2 \% \quad (6.12) \quad \text{passed}$$

11. Cross-section resistance to shear (z):

EN 1993-1-1: 6.2.6, 6.2.7

Critical combination: $[1.35 \cdot 0.85 \cdot G_k] \{1.5 \cdot Q_k\}$

Section class: **1** (Plastic design)

Critical section: $x = 0.000 \cdot L = 0.000 \cdot 60 = 0 \text{ cm}$

$$A_{V,z} = A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f = 9 \text{ cm}^2$$

$$V_{pl,Rd,z} = \frac{A_{V,z} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{9 \cdot 36}{\sqrt{3} \cdot 1} = 185 \text{ kN} \quad (6.18)$$

$$M_{x,Ed} = -9.3 \text{ kNcm}$$

$$V_{pl,T,Rd,z} = \left(\sqrt{1 - \frac{\tau_{T,xz,Ed}}{1.25 \cdot \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}}}} \right) \cdot V_{pl,Rd,z} = \left(\sqrt{1 - \frac{0.59}{1.25 \cdot \frac{36}{\sqrt{3} \cdot 1}}} \right) \cdot 185 = 183 \text{ kN} \quad (6.26)$$

$$\eta_{V_z} = \frac{|V_{z,Ed}|}{V_{pl,T,Rd,z}} = \frac{|(-76)|}{183} = 41.4 \% \quad (6.17) \quad \text{passed}$$

12. Bending-shear interaction check

EN 1993-1-1: 6.2.1, 6.2.8, 6.2.9

Critical combination for N-M-V strength interaction: $[1.35 \cdot 0.85 \cdot G_k] \{1.5 \cdot Q_k\}$

Section class: **1** (Plastic design)

Critical section: $x = 1.000 \cdot L = 1.000 \cdot 60 = 60 \text{ cm}$

$$V_{z,Ed} = -76 \text{ kN} \leq V_{pl,Rd,z} / 2 = 93 \text{ kN} \rightarrow \text{The effect of shear force on moment resistance is negligible.} \quad 6.2.8 (2)$$

$$V_{y,Ed} = 0.063 \text{ kN} \leq V_{pl,Rd,y} / 2 = 205 \text{ kN} \rightarrow \text{The effect of shear force on moment resistance is negligible.} \quad 6.2.8 (2)$$

13. Bending-axial force interaction check

EN 1993-1-1: 6.2.1, 6.2.8, 6.2.9

Critical combination for N-M-V strength interaction: $[1.35 \cdot 0.85 \cdot G_k] \{1.5 \cdot Q_k\}$

Section class: **1** (Plastic design)

Critical section: $x = 1.000 \cdot L = 1.000 \cdot 60 = 60 \text{ cm}$

$$n = \frac{|N_{Ed}|}{N_{pl,Rd}} = \frac{6.5}{924} = 0.01 = 0.71\% \leq 25\%$$

$$|N_{Ed}| = 6.5 \text{ kN} \leq N_{lim,y} = \frac{h_w \cdot t_w \cdot f_y}{2 \cdot \gamma_{M0}} = \frac{8 \cdot 0.6 \cdot 36}{2 \cdot 1} = 85 \text{ kN}$$

$$|N_{Ed}| = 6.5 \text{ kN} \leq N_{lim,z} = \frac{h_w \cdot t_w \cdot f_y}{\gamma_{M0}} = \frac{8 \cdot 0.6 \cdot 36}{1} = 170 \text{ kN}$$

$$M_{N,y,Rd} = M_{y,V,Rd} = 3700 \text{ kNcm} = 36.997 \text{ kNm}$$

$$M_{N,z,Rd} = M_{z,V,Rd} = 1826 \text{ kNcm} = 18.255 \text{ kNm}$$



$$\eta_{MN,1} = \frac{M_{y,Ed}}{M_{N,y,Rd}} = \frac{(-3534)}{3700} = 95.5 \%$$

$$\eta_{MN,2} = \frac{M_{z,Ed}}{M_{N,z,Rd}} = \frac{(-3.8)}{1826} = 0.2 \%$$

$$\alpha_{MN} = 2$$

$$\beta_{MN} = \max(5 \cdot n; 1) = \max(5 \cdot 0.01; 1) = 1$$

$$\eta_{MN,3} = \left(\frac{M_{y,Ed}}{M_{N,y,Rd}} \right)^{\alpha_{MN}} + \left(\frac{M_{z,Ed}}{M_{N,z,Rd}} \right)^{\beta_{MN}} = \left(\frac{(-3534)}{3700} \right)^2 + \left(\frac{(-3.8)}{1826} \right)^1 = 91.5 \% \quad (6.41)$$

$$\eta_{MN} = \max(\eta_{MN,1}; \eta_{MN,2}; \eta_{MN,3}; \eta_N) = \max(95.5; 0.2; 91.5; 0.7) = 95.5 \% \quad \text{passed}$$

14. Buckling resistance:

EN 1993-1-1: 6.3.1

Critical combination for N-M-Buckling interaction: **[1.35*0.85*Gk] {1.5*Qk}**

Section class: **1** (Plastic design)

Critical section: $x = 1.000 \cdot L = 1.000 \cdot 60 = 60$ cm

$$K_y = 1$$

$$K_z = 1$$

$$L_{cr,y} = K_y \cdot L = 1 \cdot 60 = 60 \text{ cm}$$

$$L_{cr,z} = K_z \cdot L = 1 \cdot 60 = 60 \text{ cm}$$

Buckling curve about the y axis: *b* Table 6.2

$$\rightarrow \alpha_y = 0.34 \quad \text{Table 6.1}$$

Buckling curve about the z axis: *c* Table 6.2

$$\rightarrow \alpha_z = 0.49 \quad \text{Table 6.1}$$

$$\lambda_{y,*} = \sqrt{\frac{A \cdot f_y}{N_{cr,y}}} = \sqrt{\frac{26 \cdot 36}{2.5882 \cdot 10^4}} = 0.19 \quad (6.50)$$

$$\lambda_{z,*} = \sqrt{\frac{A \cdot f_y}{N_{cr,z}}} = \sqrt{\frac{26 \cdot 36}{9630}} = 0.31 \quad (6.50)$$

$$\phi_y = \frac{1 + \alpha_y \cdot (\lambda_{y,*} - 0.2) + \lambda_{y,*}^2}{2} = \frac{1 + 0.34 \cdot (0.19 - 0.2) + 0.19^2}{2} = 0.516$$

$$\phi_z = \frac{1 + \alpha_z \cdot (\lambda_{z,*} - 0.2) + \lambda_{z,*}^2}{2} = \frac{1 + 0.49 \cdot (0.31 - 0.2) + 0.31^2}{2} = 0.5749$$

$$\chi_y = \min \left(\frac{1}{\phi_y + \sqrt{\phi_y^2 - \lambda_{y,*}^2}}; 1 \right) = \min \left(\frac{1}{0.516 + \sqrt{0.516^2 - 0.19^2}}; 1 \right) = 1.00 \quad (6.49)$$

$$\chi_z = \min \left(\frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_{z,*}^2}}; 1 \right) = \min \left(\frac{1}{0.5749 + \sqrt{0.5749^2 - 0.31^2}}; 1 \right) = 0.94 \quad (6.49)$$

$$\chi = \min(\chi_y; \chi_z) = \min(1.00; 0.94) = 0.94 \leq 1.0$$

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.94 \cdot 26 \cdot 36}{1} = 873 \text{ kN} \quad (6.47)$$



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$$\eta_{N_b} = \frac{|N_{Ed}|}{N_{b,Rd}} = \frac{|(-6.5)|}{873} = 0.8 \% \quad (6.46) \quad \text{passed}$$

15. Lateral-torsional buckling resistance:

[EN 1993-1-1: 6.3.2](#)

Disabled