



**ΕΛΛΗΝΙΚΗ ΔΗΜΟΚΡΑΤΙΑ
ΠΕΡΙΦΕΡΕΙΑ ΘΕΣΣΑΛΙΑΣ
ΝΟΜΟΣ ΜΑΓΝΗΣΙΑΣ
ΔΗΜΟΣ ΡΗΓΑ ΦΕΡΑΙΟΥ**

*Δ/ΝΣΗ ΤΕΧΝΙΚΩΝ ΥΠΗΡΕΣΙΩΝ &
ΠΕΡΙΒΑΛΛΟΝΤΟΣ
ΤΕΧΝΙΚΗ ΥΠΗΡΕΣΙΑ*

ΕΡΓΟ :

**ΑΝΑΒΑΘΜΙΣΗ ΤΟΥ ΕΠΑ.Λ.
ΒΕΛΕΣΤΙΝΟΥ ΤΟΥ ΔΗΜΟΥ ΡΗΓΑ
ΦΕΡΑΙΟΥ ΣΕ ΠΡΟΤΥΠΟ ΕΠΑ.Λ.**

Αρ. Μελέτης: 6 /2023

ΠΡΟΫΠΟΛΟΓΙΣΜΟΥ: 480.000,00 €

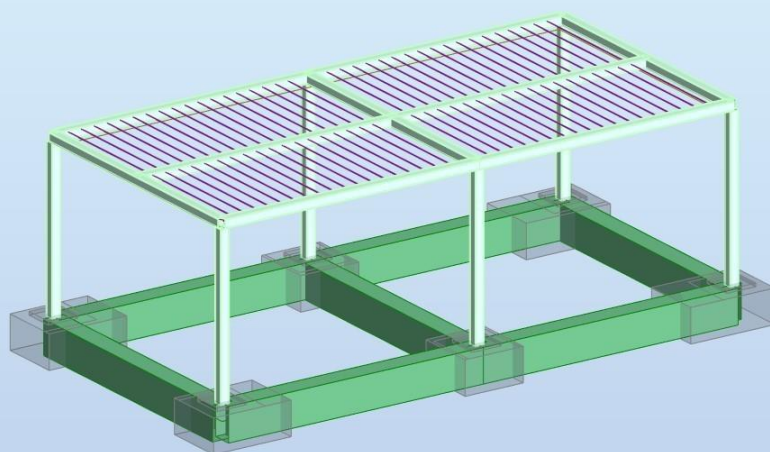
ΣΥΜΠΕΡΙΛΑΜΒΑΝΕΤΕ Ο ΦΠΑ

Αρ. Πρωτ: 530/19-01-2023

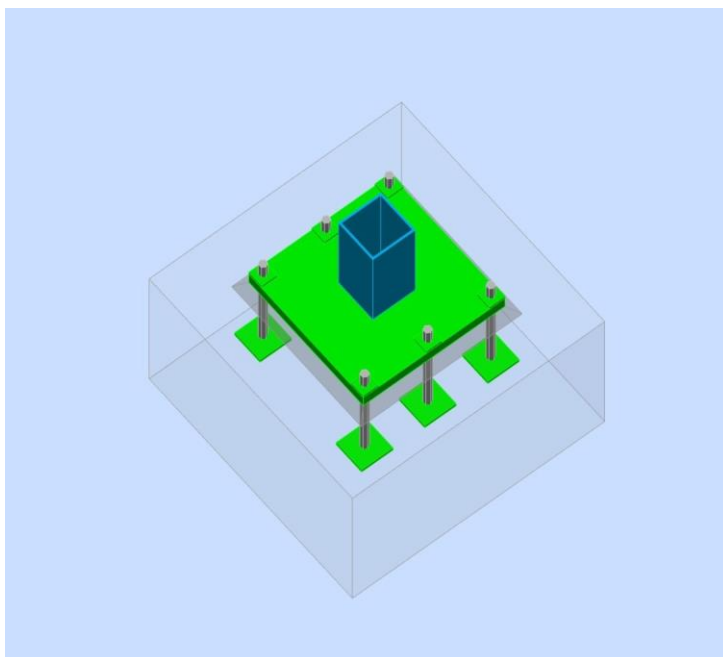
**ΧΡΗΜΑΤΟΔΟΤΗΣΗ :
ΤΑΜΕΙΟ ΑΝΑΚΑΜΨΗΣ**

**ΣΤΑΤΙΚΗ ΜΕΛΕΤΗ ΠΕΡΓΚΟΛΑΣ
(ΑΠΟΤΕΛΕΣΜΑΤΑ ΣΥΝΔΕΣΕΩΝ)**

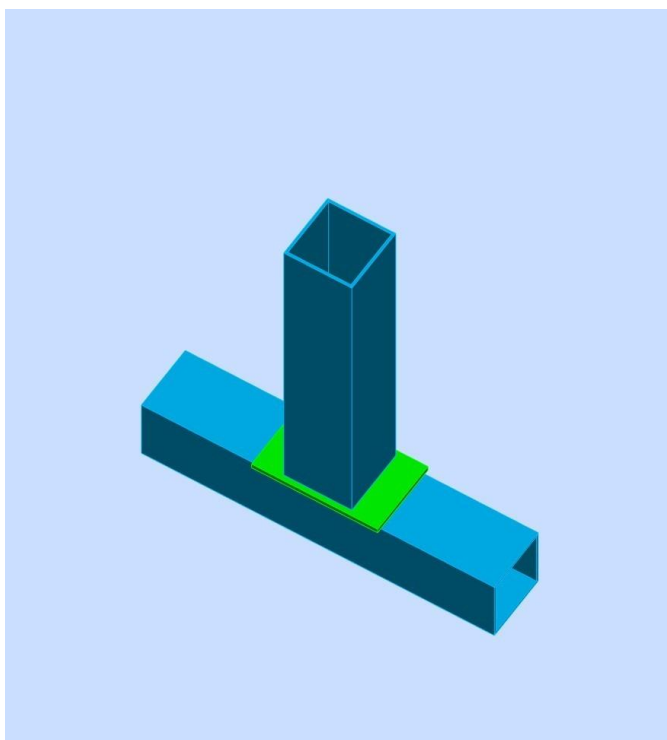
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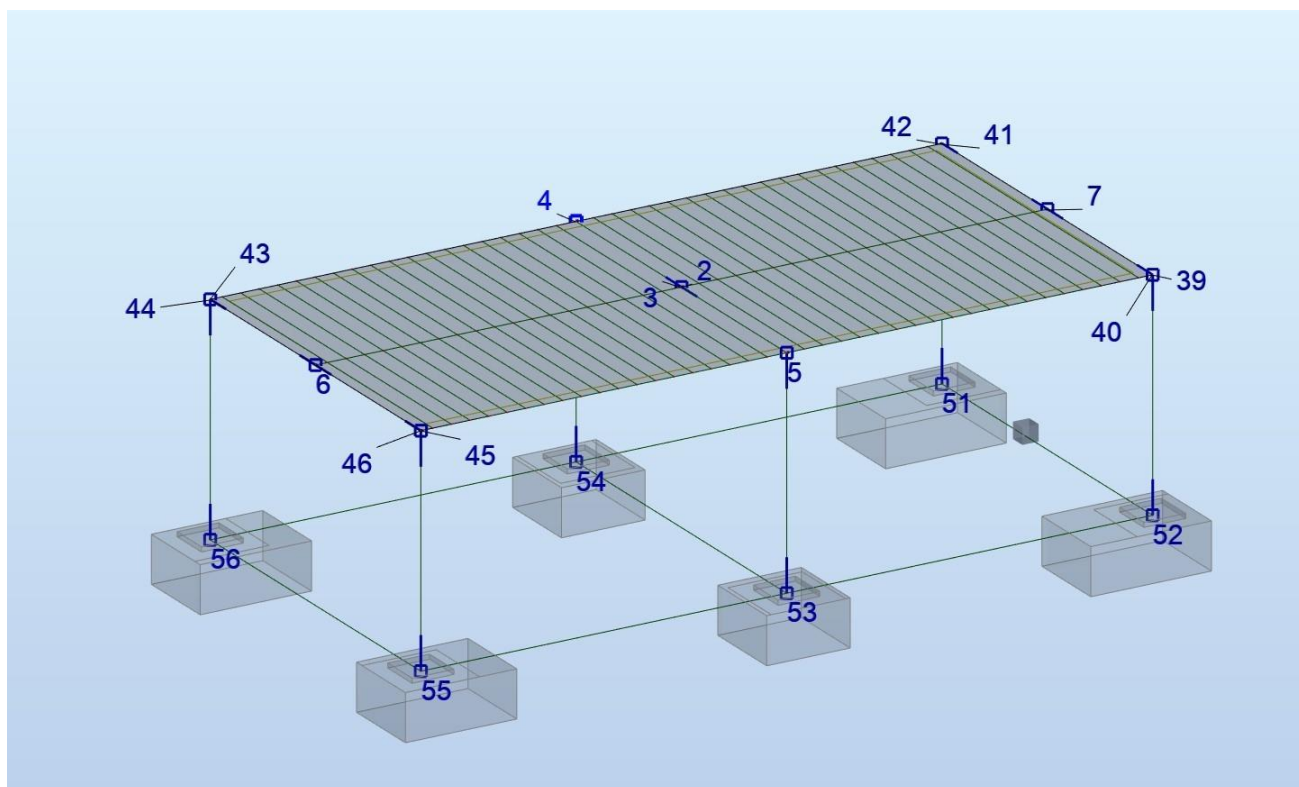
— B R30x50
— NZSE 10x15x0.2
— TCAR 135x5



ΑΞΟΝΟΜΕΤΡΙΚΗ ΑΠΕΙΚΟΝΙΣΗ ΣΥΝΔΕΣΗΣ
ΘΕΜΕΛΙΩΣΗΣ



ΑΞΟΝΟΜΕΤΡΙΚΗ ΑΠΕΙΚΟΝΙΣΗ ΣΥΝΔΕΣΗΣ
ΚΟΙΛΩΝ ΔΙΑΤΟΜΩΝ



ΑΡΙΘΜΙΣΗ ΣΥΝΔΕΣΕΩΝ

Connection Verification

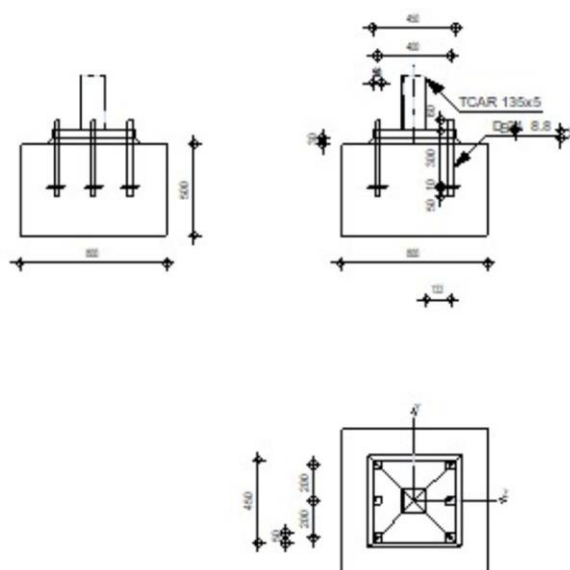


Autodesk Robot Structural Analysis Professional 2021

Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009 + CEB Design Guide:

Design of fastenings in concrete

Ratio
0,95

GENERAL

Connection no.: 52

Connection name: Fixed column base

Structure node: 1

Structure bars: 1

GEOMETRY

COLUMN

Section: TCAR 135x5

Bar no.: 1

$L_c =$	2,40	[m]	Column length
$\alpha =$	0,0	[Deg]	Inclination angle
$h_c =$	135	[mm]	Height of column section
$b_{fc} =$	135	[mm]	Width of column section
$t_{wc} =$	5	[mm]	Thickness of the web of column section
$t_{fc} =$	5	[mm]	Thickness of the flange of column section
$r_c =$	11	[mm]	Radius of column section fillet
$A_c =$	25,10	[cm ²]	Cross-sectional area of a column
$I_{yc} =$	695,00	[cm ⁴]	Moment of inertia of the column section
Material: S275			
$f_{yc} =$	275,00	[MPa]	Resistance
$f_{uc} =$	430,00	[MPa]	Yield strength of a material

COLUMN BASE

$l_{pd} =$	450	[mm]	Length
$b_{pd} =$	450	[mm]	Width
$t_{pd} =$	40	[mm]	Thickness
Material: S275			
$f_{ypd} =$	275,00	[MPa]	Resistance
$f_{upd} =$	430,00	[MPa]	Yield strength of a material

ANCHORAGE

The shear plane passes through the UNTHREADED portion of the bolt.

Class =	8.8		Anchor class
$f_{yb} =$	640,00	[MPa]	Yield strength of the anchor material
$f_{ub} =$	800,00	[MPa]	Tensile strength of the anchor material
$d =$	24	[mm]	Bolt diameter
$A_s =$	3,53	[cm ²]	Effective section area of a bolt
$A_v =$	4,52	[cm ²]	Area of bolt section
$n_H =$	2		Number of bolt columns
$n_V =$	3		Number of bolt rows
Horizontal spacing $e_{Hi} =$ 400 [mm]			
Vertical spacing $e_{Vi} =$ 200 [mm]			

Anchor dimensions

$L_1 =$	60	[mm]
$L_2 =$	300	[mm]
$L_3 =$	50	[mm]

Anchor plate

$l_p = 100$ [mm] Length
 $b_p = 100$ [mm] Width
 $t_p = 10$ [mm] Thickness
 Material: S275
 $f_y = 275,00$ [MPa] Resistance

Washer

$l_{wd} = 50$ [mm] Length
 $b_{wd} = 50$ [mm] Width
 $t_{wd} = 10$ [mm] Thickness

MATERIAL FACTORS

$\gamma_{M0} = 1,00$ Partial safety factor
 $\gamma_{M2} = 1,25$ Partial safety factor
 $\gamma_C = 1,50$ Partial safety factor

SPREAD FOOTING

$L = 800$ [mm] Spread footing length
 $B = 800$ [mm] Spread footing width
 $H = 500$ [mm] Spread footing height

Concrete

Class C25/30

$f_{ck} = 25,00$ [MPa] Characteristic resistance for compression

Grout layer

$t_g = 30$ [mm] Thickness of leveling layer (grout)
 $f_{ck,g} = 12,00$ [MPa] Characteristic resistance for compression
 $C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete

WELDS

$a_p = 5$ [mm] Footing plate of the column base

LOADS

Case: 18: 1 * X -1 * Y SQRT(11*1.00;12*-1.00)

$N_{j,Ed} = -3,03$ [kN] Axial force
 $V_{j,Ed,y} = 6,73$ [kN] Shear force
 $V_{j,Ed,z} = 6,35$ [kN] Shear force
 $M_{j,Ed,y} = 9,12$ [kN*m] Bending moment
 $M_{j,Ed,z} = 10,83$ [kN*m] Bending moment

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

$f_{cd} = 16,67$ [MPa] Design compressive resistance EN 1992-1:[3.1.6.(1)]
 $f_j = 19,75$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]
 $c = t_p \sqrt{(f_{yp}/(3*f_j*\gamma_{M0}))}$
 $c = 86$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]
 $b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]
 $l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]
 $A_{c0} = 545,02$ [cm²] Area of the joint between the base plate and the foundation EN 1992-1:[6.7.(3)]
 $A_{c1} = 4256,09$ [cm²] Maximum design area of load distribution EN 1992-1:[6.7.(3)]
 $F_{rd,u} = A_{c0} * f_{cd} * \sqrt{(A_{c1}/A_{c0})} \leq 3 * A_{c0} * f_{cd}$
 $F_{rd,u} = 2538,40$ [kN] Bearing resistance of concrete EN 1992-1:[6.7.(3)]
 $\beta_j = 0,67$ Reduction factor for compression [6.2.5.(7)]
 $f_{jd} = \beta_j * F_{rd,u} / (b_{eff} * l_{eff})$
 $f_{jd} = 31,05$ [MPa] Design bearing resistance [6.2.5.(7)]
 $A_{c,n} = 877,03$ [cm²] Bearing area for compression [6.2.8.2.(1)]
 $A_{c,y} = 438,51$ [cm²] Bearing area for bending My [6.2.8.3.(1)]
 $A_{c,z} = 438,51$ [cm²] Bearing area for bending Mz [6.2.8.3.(1)]
 $F_{c,Rd,i} = A_{c,i} * f_{jd}$
 $F_{c,Rd,n} = 2723,13$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]
 $F_{c,Rd,y} = 1361,57$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]
 $F_{c,Rd,z} = 1361,57$ [kN] Bearing resistance of concrete for bending Mz [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL = 1,00$ Section class EN 1993-1-1:[5.5.2]
 $W_{pl,y} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:[6.2.5.(2)]
 $M_{c,Rd,y} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]
 $h_{f,y} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]
 $F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$

$$F_{c,fc,Rd,y} = 217,81 \text{ [kN]} \quad \text{Resistance of the compressed flange and web} \quad [6.2.6.7.(1)]$$

$$W_{pl,z} = 102,96 \text{ [cm}^3\text{]} \quad \text{Plastic section modulus} \quad \text{EN1993-1-1:[6.2.5.(2)]}$$

$$M_{c,Rd,z} = 28,31 \text{ [kN*m]} \quad \text{Design resistance of the section for bending} \quad \text{EN1993-1-1:[6.2.5]}$$

$$h_{f,z} = 130 \text{ [mm]} \quad \text{Distance between the centroids of flanges} \quad [6.2.6.7.(1)]$$

$$F_{c,fc,Rd,z} = M_{c,Rd,z} / h_{f,z}$$

$$F_{c,fc,Rd,z} = 217,81 \text{ [kN]} \quad \text{Resistance of the compressed flange and web} \quad [6.2.6.7.(1)]$$

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$$N_{j,Rd} = F_{c,Rd,n}$$

$$N_{j,Rd} = 2723,13 \text{ [kN]} \quad \text{Resistance of a spread footing for axial compression} \quad [6.2.8.2.(1)]$$

$$F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$$

$$F_{C,Rd,y} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

$$F_{C,Rd,z} = \min(F_{c,Rd,z}, F_{c,fc,Rd,z})$$

$$F_{C,Rd,z} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

TENSION ZONE

STEEL FAILURE

$$A_b = 3,53 \text{ [cm}^2\text{]} \quad \text{Effective anchor area} \quad [\text{Table 3.4}]$$

$$f_{ub} = 800,00 \text{ [MPa]} \quad \text{Tensile strength of the anchor material} \quad [\text{Table 3.4}]$$

$$\text{Beta} = 0,85 \quad \text{Reduction factor of anchor resistance} \quad [3.6.1.(3)]$$

$$F_{t,Rd,s1} = \text{beta} * 0.9 * f_{ub} * A_b / \gamma_{M2}$$

$$F_{t,Rd,s1} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad [\text{Table 3.4}]$$

$$\gamma_{Ms} = 1,20 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.2]}$$

$$f_{yb} = 640,00 \text{ [MPa]} \quad \text{Yield strength of the anchor material} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s2} = f_{yb} * A_b / \gamma_{Ms}$$

$$F_{t,Rd,s2} = 188,27 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$$

$$F_{t,Rd,s} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure}$$

PULL-OUT FAILURE

$$f_{ck} = 25,00 \text{ [MPa]} \quad \text{Characteristic compressive strength of concrete} \quad \text{EN 1992-1:[3.1.2]}$$

$$A_h = 95,48 \text{ [cm}^2\text{]} \quad \text{Bearing area of the head} \quad \text{CEB [15.1.2.3]}$$

$$p_k = 175,00 \text{ [MPa]} \quad \text{Characteristic strength of concrete (pull-out)} \quad \text{CEB [15.1.2.3]}$$

$$\gamma_{Mp} = 2,16 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.1]}$$

$$F_{t,Rd,p} = p_k * A_h / \gamma_{Mp}$$

$$F_{t,Rd,p} = 828,79 \text{ [kN]} \quad \text{Design uplift capacity} \quad \text{CEB [9.2.3]}$$

CONCRETE CONE FAILURE

$h_{ef} = 133$ [mm] Effective anchorage depth CEB [9.2.4]

$$N_{Rk,c}^0 = 9.0[N^{0.5}/mm^{0.5}] * f_{ck}^{0.5} * h_{ef}^{1.5}$$

$N_{Rk,c}^0 = 69,28$ [kN] Characteristic resistance of an anchor CEB [9.2.4]

$s_{cr,N} = 400$ [mm] Critical width of the concrete cone CEB [9.2.4]

$c_{cr,N} = 200$ [mm] Critical edge distance CEB [9.2.4]

$A_{c,N0} = 1600,00$ [cm²] Maximum area of concrete cone CEB [9.2.4]

$A_{c,N} = 1600,00$ [cm²] Actual area of concrete cone CEB [9.2.4]

$$\psi_{A,N} = A_{c,N}/A_{c,N0}$$

$\psi_{A,N} = 1,00$ Factor related to anchor spacing and edge distance CEB [9.2.4]

$c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.4]

$$\psi_{s,N} = 0.7 + 0.3 * c/c_{cr,N} \leq 1.0$$

$\psi_{s,N} = 1,00$ Factor taking account the influence of edges of the concrete member on the distribution of stresses

$\psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors

$$\psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$$

$\psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.4]

$\psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.4]

$\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3]

$$F_{t,Rd,c} = N_{Rk,c}^0 * \psi_{A,N} * \psi_{s,N} * \psi_{ec,N} * \psi_{re,N} * \psi_{ucr,N} / \gamma_{Mc}$$

$F_{t,Rd,c} = 32,08$ [kN] Design anchor resistance to concrete cone failure EN 1992-1-1:8.4.2.(2)

SPLITTING FAILURE

$h_{ef} = 270$ [mm] Effective anchorage depth CEB [9.2.5]

$$N_{Rk,c}^0 = 9.0[N^{0.5}/mm^{0.5}] * f_{ck}^{0.5} * h_{ef}^{1.5}$$

$N_{Rk,c}^0 = 199,64$ [kN] Design uplift capacity CEB [9.2.5]

$s_{cr,N} = 540$ [mm] Critical width of the concrete cone CEB [9.2.5]

$c_{cr,N} = 270$ [mm] Critical edge distance CEB [9.2.5]

$A_{c,N0} = 2916,00$ [cm²] Maximum area of concrete cone CEB [9.2.5]

$A_{c,N} = 1880,00$ [cm²] Actual area of concrete cone CEB [9.2.5]

$$\psi_{A,N} = A_{c,N}/A_{c,N0}$$

$\psi_{A,N} = 0,64$ Factor related to anchor spacing and edge distance CEB [9.2.5]

$c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.5]

$$\psi_{s,N} = 0.7 + 0.3 * c/c_{cr,N} \leq 1.0$$

$\psi_{s,N} = 0,92$ Factor taking account the influence of edges of the concrete member on the distribution of stresses

$\psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors

$$\psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$$

$\psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.5]

$\psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]

$$\psi_{h,N} = (h/(2 \cdot h_{ef}))^{2/3} \leq 1.2$$

$$\psi_{h,N} = 0,95 \quad \text{Coeff. related to the foundation height} \quad \text{CEB [9.2.5]}$$

$$\gamma_{M,sp} = 2,16 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.1]}$$

$$F_{t,Rd,sp} = N_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{ucr,N} \cdot \psi_{h,N} / \gamma_{M,sp}$$

$$F_{t,Rd,sp} = 52,21 \quad [\text{kN}] \quad \text{Design anchor resistance to splitting of concrete} \quad \text{CEB [9.2.5]}$$

TENSILE RESISTANCE OF AN ANCHOR

$$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$$

$$F_{t,Rd} = 32,08 \quad [\text{kN}] \quad \text{Tensile resistance of an anchor}$$

BENDING OF THE BASE PLATE

Bending moment $M_{j,Ed,y}$

$$l_{eff,1} = 225 \quad [\text{mm}] \quad \text{Effective length for a single bolt for mode 1} \quad [6.2.6.5]$$

$$l_{eff,2} = 225 \quad [\text{mm}] \quad \text{Effective length for a single bolt for mode 2} \quad [6.2.6.5]$$

$$m = 127 \quad [\text{mm}] \quad \text{Distance of a bolt from the stiffening edge} \quad [6.2.6.5]$$

$$M_{pl,1,Rd} = 24,75 \quad [\text{kN} \cdot \text{m}] \quad \text{Plastic resistance of a plate for mode 1} \quad [6.2.4]$$

$$M_{pl,2,Rd} = 24,75 \quad [\text{kN} \cdot \text{m}] \quad \text{Plastic resistance of a plate for mode 2} \quad [6.2.4]$$

$$F_{T,1,Rd} = 780,49 \quad [\text{kN}] \quad \text{Resistance of a plate for mode 1} \quad [6.2.4]$$

$$F_{T,2,Rd} = 341,84 \quad [\text{kN}] \quad \text{Resistance of a plate for mode 2} \quad [6.2.4]$$

$$F_{T,3,Rd} = 96,23 \quad [\text{kN}] \quad \text{Resistance of a plate for mode 3} \quad [6.2.4]$$

$$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$$

$$F_{t,pl,Rd,y} = 96,23 \quad [\text{kN}] \quad \text{Tension resistance of a plate} \quad [6.2.4]$$

Bending moment $M_{j,Ed,z}$

$$l_{eff,1} = 225 \quad [\text{mm}] \quad \text{Effective length for a single bolt for mode 1} \quad [6.2.6.5]$$

$$l_{eff,2} = 225 \quad [\text{mm}] \quad \text{Effective length for a single bolt for mode 2} \quad [6.2.6.5]$$

$$m = 187 \quad [\text{mm}] \quad \text{Distance of a bolt from the stiffening edge} \quad [6.2.6.5]$$

$$M_{pl,1,Rd} = 24,75 \quad [\text{kN} \cdot \text{m}] \quad \text{Plastic resistance of a plate for mode 1} \quad [6.2.4]$$

$$M_{pl,2,Rd} = 24,75 \quad [\text{kN} \cdot \text{m}] \quad \text{Plastic resistance of a plate for mode 2} \quad [6.2.4]$$

$$F_{T,1,Rd} = 528,33 \quad [\text{kN}] \quad \text{Resistance of a plate for mode 1} \quad [6.2.4]$$

$$F_{T,2,Rd} = 232,42 \quad [\text{kN}] \quad \text{Resistance of a plate for mode 2} \quad [6.2.4]$$

$$F_{T,3,Rd} = 64,15 \quad [\text{kN}] \quad \text{Resistance of a plate for mode 3} \quad [6.2.4]$$

$$F_{t,pl,Rd,z} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$$

$$F_{t,pl,Rd,z} = 64,15 \quad [\text{kN}] \quad \text{Tension resistance of a plate} \quad [6.2.4]$$

TENSILE RESISTANCE OF A COLUMN WEB

Bending moment $M_{j,Ed,y}$

$$t_{wc} = 5 \quad [\text{mm}] \quad \text{Effective thickness of the column web} \quad [6.2.6.3.(8)]$$

$$b_{eff,t,wc} = 225 \quad [\text{mm}] \quad \text{Effective width of the web for tension} \quad [6.2.6.3.(2)]$$

$$A_{vc} = 12,55 \quad [\text{cm}^2] \quad \text{Shear area} \quad \text{EN1993-1-1:[6.2.6.(3)]}$$

$$\omega = 0,44 \quad \text{Reduction factor for interaction with shear} \quad [6.2.6.3.(4)]$$

$$F_{t,wc,Rd,y} = \omega b_{eff,t,wc} t_{wc} f_{yc} / \gamma_{M0}$$

$$F_{t,wc,Rd,y} = 271,90 \text{ [kN]} \quad \text{Column web resistance} \quad [6.2.6.3.(1)]$$

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$$F_{T,Rd,y} = \min(F_{t,pl,Rd,y}, F_{t,wc,Rd,y})$$

$$F_{T,Rd,y} = 96,23 \text{ [kN]} \quad \text{Resistance of a column base in the tension zone} \quad [6.2.8.3]$$

$$F_{T,Rd,z} = F_{t,pl,Rd,z}$$

$$F_{T,Rd,z} = 64,15 \text{ [kN]} \quad \text{Resistance of a column base in the tension zone} \quad [6.2.8.3]$$

CONNECTION CAPACITY CHECK

$$N_{j,Ed} / N_{j,Rd} \leq 1,0 \text{ (6.24)} \quad 0,00 < 1,00 \quad \text{verified} \quad (0,00)$$

$$e_y = 3015 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 84 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 28,07 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \text{ (6.23)} \quad 0,32 < 1,00 \quad \text{verified} \quad (0,32)$$

$$e_z = 3579 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,z} = 65 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,z} \quad [6.2.8.1.(2)]$$

$$z_{t,z} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,z} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,z} = 17,31 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \text{ (6.23)} \quad 0,63 < 1,00 \quad \text{verified} \quad (0,63)$$

$$M_{j,Ed,y} / M_{j,Rd,y} + M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \quad 0,95 < 1,00 \quad \text{verified} \quad (0,95)$$

SHEAR

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

Shear force $V_{j,Ed,y}$

$$\alpha_{d,y} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,y} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,y} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,y} = k_{1,y} \alpha_{b,y} f_{up} d^2 t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,y} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

Shear force $V_{j,Ed,z}$

$\alpha_{d,z} = 0,32$ Coeff. taking account of the bolt position - in the direction of shear [Table 3.4]

$\alpha_{b,z} = 0,32$ Coeff. for resistance calculation $F_{1,vb,Rd}$ [Table 3.4]

$k_{1,z} = 0,99$ Coeff. taking account of the bolt position - perpendicularly to the direction of shear [Table 3.4]

$$F_{1,vb,Rd,z} = k_{1,z} \cdot \alpha_{b,z} \cdot f_{up} \cdot d \cdot t_p / \gamma_{M2}$$

$F_{1,vb,Rd,z} = 105,03$ [kN] Resistance of an anchor bolt for bearing pressure onto the base plate [6.2.2.(7)]

SHEAR OF AN ANCHOR BOLT

$\alpha_b = 0,25$ Coeff. for resistance calculation $F_{2,vb,Rd}$ [6.2.2.(7)]

$A_{vb} = 4,52$ [cm²] Area of bolt section [6.2.2.(7)]

$f_{ub} = 800,00$ [MPa] Tensile strength of the anchor material [6.2.2.(7)]

$\gamma_{M2} = 1,25$ Partial safety factor [6.2.2.(7)]

$$F_{2,vb,Rd} = \alpha_b \cdot f_{ub} \cdot A_{vb} / \gamma_{M2}$$

$F_{2,vb,Rd} = 71,80$ [kN] Shear resistance of a bolt - without lever arm [6.2.2.(7)]

$\alpha_M = 2,00$ Factor related to the fastening of an anchor in the foundation CEB [9.3.2.2]

$M_{Rk,s} = 1,10$ [kN*m] Characteristic bending resistance of an anchor CEB [9.3.2.2]

$l_{sm} = 62$ [mm] Lever arm length CEB [9.3.2.2]

$\gamma_{Ms} = 1,20$ Partial safety factor CEB [3.2.3.2]

$$F_{v,Rd,sm} = \alpha_M \cdot M_{Rk,s} / (l_{sm} \cdot \gamma_{Ms})$$

$F_{v,Rd,sm} = 29,45$ [kN] Shear resistance of a bolt - with lever arm CEB [9.3.1]

CONCRETE PRY-OUT FAILURE

$N_{Rk,c} = 69,28$ [kN] Design uplift capacity CEB [9.2.4]

$k_3 = 2,00$ Factor related to the anchor length CEB [9.3.3]

$\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.1]

$$F_{v,Rd,cp} = k_3 \cdot N_{Rk,c} / \gamma_{Mc}$$

$F_{v,Rd,cp} = 64,15$ [kN] Concrete resistance for pry-out failure CEB [9.3.1]

CONCRETE EDGE FAILURE

Shear force $V_{j,Ed,y}$

$V_{Rk,c,y}^0 = 70,02$ [kN] Characteristic resistance of an anchor

$\psi_{A,V,y} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,y} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,y} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,y} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,y} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,y} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$$F_{v,Rd,c,y} = V_{Rk,c,y}^0 \cdot \psi_{A,V,y} \cdot \psi_{h,V,y} \cdot \psi_{s,V,y} \cdot \psi_{ec,V,y} \cdot \psi_{\alpha,V,y} \cdot \psi_{ucr,V,y} / \gamma_{Mc}$$

$F_{v,Rd,c,y} = 19,45$ [kN] Concrete resistance for edge failure CEB [9.3.1]

Shear force $V_{j,Ed,z}$
 $V_{Rk,c,z}^0 = 70,02$ [kN] Characteristic resistance of an anchor

 $\Psi_{A,V,z} = 0,67$ Factor related to anchor spacing and edge distance

 $\Psi_{h,V,z} = 1,00$ Factor related to the foundation thickness

 $\Psi_{s,V,z} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

 $\Psi_{ec,V,z} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

 $\Psi_{\alpha,V,z} = 1,00$ Factor related to the angle at which the shear load is applied

 $\Psi_{ucr,V,z} = 1,00$ Factor related to the type of edge reinforcement used

 $\gamma_{Mc} = 2,16$ Partial safety factor

 $F_{v,Rd,c,z} = V_{Rk,c,z}^0 \cdot \Psi_{A,V,z} \cdot \Psi_{h,V,z} \cdot \Psi_{s,V,z} \cdot \Psi_{ec,V,z} \cdot \Psi_{\alpha,V,z} \cdot \Psi_{ucr,V,z} / \gamma_{Mc}$
 $F_{v,Rd,c,z} = 19,45$ [kN] Concrete resistance for edge failure CEB [9.3.1]
SPLITTING RESISTANCE
 $C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete [6.2.2.(6)]

 $N_{c,Ed} = 3,03$ [kN] Compressive force [6.2.2.(6)]

 $F_{f,Rd} = C_{f,d} \cdot N_{c,Ed}$
 $F_{f,Rd} = 0,91$ [kN] Slip resistance [6.2.2.(6)]
SHEAR CHECK
 $V_{j,Rd,y} = n_b \cdot \min(F_{1,vb,Rd,y}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{f,Rd}$
 $V_{j,Rd,y} = 117,61$ [kN] Connection resistance for shear CEB [9.3.1]

 $V_{j,Ed,y} / V_{j,Rd,y} \leq 1,0$ $0,06 < 1,00$ verified (0,06)

 $V_{j,Rd,z} = n_b \cdot \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$
 $V_{j,Rd,z} = 117,61$ [kN] Connection resistance for shear CEB [9.3.1]

 $V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,05 < 1,00$ verified (0,05)

 $V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,11 < 1,00$ verified (0,11)
WELDS BETWEEN THE COLUMN AND THE BASE PLATE
 $\sigma_{\perp} = 118,59$ [MPa] Normal stress in a weld [4.5.3.(7)]

 $\tau_{\perp} = 118,59$ [MPa] Perpendicular tangent stress [4.5.3.(7)]

 $\tau_{yII} = 4,99$ [MPa] Tangent stress parallel to $V_{j,Ed,y}$ [4.5.3.(7)]

 $\tau_{zII} = 4,70$ [MPa] Tangent stress parallel to $V_{j,Ed,z}$ [4.5.3.(7)]

 $\beta_w = 0,85$ Resistance-dependent coefficient [4.5.3.(7)]

$\sigma_{\perp} / (0.9 \cdot f_u / \gamma_{M2}) \leq 1.0$ (4.1)	0,38 < 1,00	verified	(0,38)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{yII}^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1.0$ (4.1)	0,59 < 1,00	verified	(0,59)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{zII}^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1.0$ (4.1)	0,55 < 1,00	verified	(0,55)

CONNECTION STIFFNESS

Bending moment $M_{j,Ed,y}$

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff}) / (1.275 \cdot E)}$$

$k_{13,y} = 27$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,y} = 6$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,y} = 1.6 \cdot A_b / L_b$$

$k_{16,y} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,y} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,y} = 24473,52$ [kN*m] Initial rotational stiffness [Table 6.12]

$S_{j,rig,y} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,y} \geq S_{j,rig,y}$ RIGID [5.2.2.5.(2)]

Bending moment $M_{j,Ed,z}$

$$k_{13,z} = E_c \cdot \sqrt{(A_{c,z}) / (1.275 \cdot E)}$$

$k_{13,z} = 24$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,z} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,z} = 2$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,z} = 1.6 \cdot A_b / L_b$$

$k_{16,z} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,z} =$	0,53	Column slenderness	[5.2.2.5.(2)]
$S_{j,ini,z} =$	13846,82 [kN*m]	Initial rotational stiffness	[6.3.1.(4)]
$S_{j,rig,z} =$	18243,75 [kN*m]	Stiffness of a rigid connection	[5.2.2.5]
$S_{j,ini,z} < S_{j,rig,z}$	SEMI-RIGID		[5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

REMARKS

Horizontal distance bolt-plate edge is too small. 25 [mm] < 29 [mm]

Connection conforms to the code**Ratio 0,95**



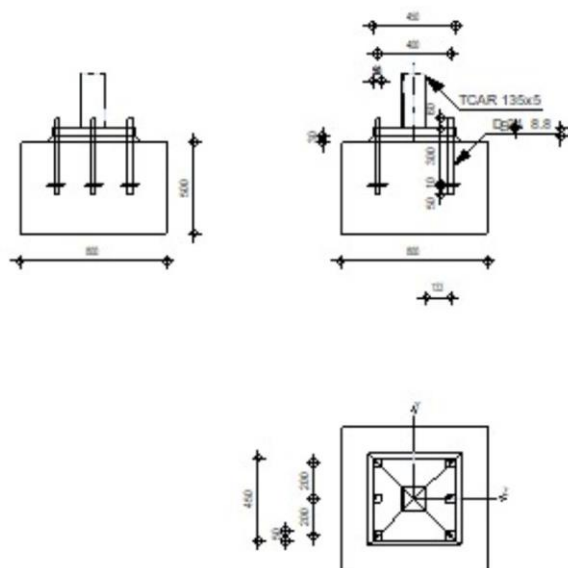
Autodesk Robot Structural Analysis Professional 2021

Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009 + CEB Design Guide:

Design of fastenings in concrete

OK

Ratio
0,89**GENERAL**

Connection no.: 53
 Connection name: Fixed column base
 Structure node: 11
 Structure bars: 5

GEOMETRY**COLUMN**

Section: TCAR 135x5

Bar no.: 5

$L_c =$	2,40	[m]	Column length
$\alpha =$	0,0	[Deg]	Inclination angle
$h_c =$	135	[mm]	Height of column section
$b_{fc} =$	135	[mm]	Width of column section
$t_{wc} =$	5	[mm]	Thickness of the web of column section
$t_{fc} =$	5	[mm]	Thickness of the flange of column section
$r_c =$	11	[mm]	Radius of column section fillet
$A_c =$	25,10	[cm ²]	Cross-sectional area of a column
$I_{yc} =$	695,00	[cm ⁴]	Moment of inertia of the column section

Material: S275

 $f_{yc} = 275,00$ [MPa] Resistance $f_{uc} = 430,00$ [MPa] Yield strength of a material**COLUMN BASE** $l_{pd} = 450$ [mm] Length $b_{pd} = 450$ [mm] Width $t_{pd} = 40$ [mm] Thickness

Material: S275

 $f_{ypd} = 275,00$ [MPa] Resistance $f_{upd} = 430,00$ [MPa] Yield strength of a material**ANCHORAGE**

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 8.8 Anchor class

 $f_{yb} = 640,00$ [MPa] Yield strength of the anchor material $f_{ub} = 800,00$ [MPa] Tensile strength of the anchor material $d = 24$ [mm] Bolt diameter $A_s = 3,53$ [cm²] Effective section area of a bolt $A_v = 4,52$ [cm²] Area of bolt section $n_H = 2$ Number of bolt columns $n_V = 3$ Number of bolt rowsHorizontal spacing $e_{Hi} = 400$ [mm]Vertical spacing $e_{Vi} = 200$ [mm]**Anchor dimensions** $L_1 = 60$ [mm] $L_2 = 300$ [mm] $L_3 = 50$ [mm]**Anchor plate** $l_p = 100$ [mm] Length $b_p = 100$ [mm] Width $t_p = 10$ [mm] Thickness

Material: S275

 $f_y = 275,00$ [MPa] Resistance**Washer**

$l_{wd} = 50$ [mm] Length
 $b_{wd} = 50$ [mm] Width
 $t_{wd} = 10$ [mm] Thickness

MATERIAL FACTORS

$\gamma_{M0} = 1,00$ Partial safety factor
 $\gamma_{M2} = 1,25$ Partial safety factor
 $\gamma_C = 1,50$ Partial safety factor

SPREAD FOOTING

$L = 800$ [mm] Spread footing length
 $B = 800$ [mm] Spread footing width
 $H = 500$ [mm] Spread footing height

Concrete

Class C25/30

$f_{ck} = 25,00$ [MPa] Characteristic resistance for compression

Grout layer

$t_g = 30$ [mm] Thickness of leveling layer (grout)

$f_{ck,g} = 12,00$ [MPa] Characteristic resistance for compression

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete

WELDS

$a_p = 5$ [mm] Footing plate of the column base

LOADS

Case: 18: 1 * X -1 * Y SQRT(11*1.00;12*-1.00)

$N_{j,Ed} = -0,18$ [kN] Axial force
 $V_{j,Ed,y} = 3,55$ [kN] Shear force
 $V_{j,Ed,z} = 4,46$ [kN] Shear force
 $M_{j,Ed,y} = 10,70$ [kN*m] Bending moment
 $M_{j,Ed,z} = 8,52$ [kN*m] Bending moment

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

$f_{cd} = 16,67$ [MPa] Design compressive resistance EN 1992-1-1:3.1.6.(1)

$f_j = 19,75$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

$$c = t_p \sqrt{(f_{yp} / (3 * f_j * \gamma_{M0}))}$$

$c = 86$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$A_{c0} = 545,02$ [cm²] Area of the joint between the base plate and the foundation EN 1992-1-1:6.7.(3)

$A_{c1} = 4256,09$ [cm²] Maximum design area of load distribution EN 1992-1-1:6.7.(3)

$$F_{rd,u} = A_{c0} * f_{cd} * \sqrt{(A_{c1} / A_{c0})} \leq 3 * A_{c0} * f_{cd}$$

$F_{rd,u} = 2538,40$ [kN] Bearing resistance of concrete EN 1992-1-1:6.7.(3)

$\beta_j = 0,67$ Reduction factor for compression [6.2.5.(7)]

$$f_{jd} = \beta_j * F_{rd,u} / (b_{eff} * l_{eff})$$

$f_{jd} = 31,05$ [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n} = 877,03$ [cm²] Bearing area for compression [6.2.8.2.(1)]

$A_{c,y} = 438,51$ [cm²] Bearing area for bending My [6.2.8.3.(1)]

$A_{c,z} = 438,51$ [cm²] Bearing area for bending Mz [6.2.8.3.(1)]

$$F_{c,Rd,i} = A_{c,i} * f_{jd}$$

$F_{c,Rd,n} = 2723,13$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]

$F_{c,Rd,y} = 1361,57$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

$F_{c,Rd,z} = 1361,57$ [kN] Bearing resistance of concrete for bending Mz [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL = 1,00$ Section class EN 1993-1-1:5.5.2

$W_{pl,y} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,y} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,y} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$$

$F_{c,fc,Rd,y} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

$W_{pl,z} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,z} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,z} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,z} = M_{c,Rd,z} / h_{f,z}$$

$F_{c,fc,Rd,z} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$$N_{j,Rd} = F_{c,Rd,n}$$

$N_{j,Rd} = 2723,13$ [kN] Resistance of a spread footing for axial compression [6.2.8.2.(1)]

$$F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$$

$$F_{C,Rd,y} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

$$F_{C,Rd,z} = \min(F_{c,Rd,z}, F_{c,fc,Rd,z})$$

$$F_{C,Rd,z} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

TENSION ZONE

STEEL FAILURE

$$A_b = 3,53 \text{ [cm}^2\text{]} \quad \text{Effective anchor area} \quad [\text{Table 3.4}]$$

$$f_{ub} = 800,00 \text{ [MPa]} \quad \text{Tensile strength of the anchor material} \quad [\text{Table 3.4}]$$

$$\text{Beta} = 0,85 \quad \text{Reduction factor of anchor resistance} \quad [3.6.1.(3)]$$

$$F_{t,Rd,s1} = \text{beta} \cdot 0.9 \cdot f_{ub} \cdot A_b / \gamma_{M2}$$

$$F_{t,Rd,s1} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad [\text{Table 3.4}]$$

$$\gamma_{Ms} = 1,20 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.2]}$$

$$f_{yb} = 640,00 \text{ [MPa]} \quad \text{Yield strength of the anchor material} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s2} = f_{yb} \cdot A_b / \gamma_{Ms}$$

$$F_{t,Rd,s2} = 188,27 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$$

$$F_{t,Rd,s} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure}$$

PULL-OUT FAILURE

$$f_{ck} = 25,00 \text{ [MPa]} \quad \text{Characteristic compressive strength of concrete} \quad \text{EN 1992-1:[3.1.2]}$$

$$A_h = 95,48 \text{ [cm}^2\text{]} \quad \text{Bearing area of the head} \quad \text{CEB [15.1.2.3]}$$

$$p_k = 175,00 \text{ [MPa]} \quad \text{Characteristic strength of concrete (pull-out)} \quad \text{CEB [15.1.2.3]}$$

$$\gamma_{Mp} = 2,16 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.1]}$$

$$F_{t,Rd,p} = p_k \cdot A_h / \gamma_{Mp}$$

$$F_{t,Rd,p} = 828,79 \text{ [kN]} \quad \text{Design uplift capacity} \quad \text{CEB [9.2.3]}$$

CONCRETE CONE FAILURE

$$h_{ef} = 133 \text{ [mm]} \quad \text{Effective anchorage depth} \quad \text{CEB [9.2.4]}$$

$$N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$$

$$N_{Rk,c}^0 = 69,28 \text{ [kN]} \quad \text{Characteristic resistance of an anchor} \quad \text{CEB [9.2.4]}$$

$$s_{cr,N} = 400 \text{ [mm]} \quad \text{Critical width of the concrete cone} \quad \text{CEB [9.2.4]}$$

$$c_{cr,N} = 200 \text{ [mm]} \quad \text{Critical edge distance} \quad \text{CEB [9.2.4]}$$

$$A_{c,N0} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Maximum area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$A_{c,N} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Actual area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$\psi_{A,N} = A_{c,N} / A_{c,N0}$$

$\Psi_{A,N} = 1,00$ Factor related to anchor spacing and edge distance CEB [9.2.4]
 $c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.4]
 $\Psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$
 $\Psi_{s,N} = 1,00$ Factor taking account the influence of edges of the concrete member on the distribution of stresses
 $\Psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors
 $\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$
 $\Psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.5]
 $\Psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]
 $\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.1]
 $F_{t,Rd,c} = N_{Rk,c} \cdot \Psi_{A,N} \cdot \Psi_{s,N} \cdot \Psi_{ec,N} \cdot \Psi_{re,N} \cdot \Psi_{ucr,N} / \gamma_{Mc}$
 $F_{t,Rd,c} = 32,08$ [kN] Design anchor resistance to concrete cone failure EN 1992-1-1:8.4.2.(2)

SPLITTING FAILURE

$h_{ef} = 270$ [mm] Effective anchorage depth CEB [9.2.5]
 $N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$
 $N_{Rk,c}^0 = 199,64$ [kN] Design uplift capacity CEB [9.2.5]
 $s_{cr,N} = 540$ [mm] Critical width of the concrete cone CEB [9.2.5]
 $c_{cr,N} = 270$ [mm] Critical edge distance CEB [9.2.5]
 $A_{c,N0} = 2916,00$ [cm²] Maximum area of concrete cone CEB [9.2.5]
 $A_{c,N} = 1880,00$ [cm²] Actual area of concrete cone CEB [9.2.5]
 $\Psi_{A,N} = A_{c,N}/A_{c,N0}$
 $\Psi_{A,N} = 0,64$ Factor related to anchor spacing and edge distance CEB [9.2.5]
 $c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.5]
 $\Psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$
 $\Psi_{s,N} = 0,92$ Factor taking account the influence of edges of the concrete member on the distribution of stresses
 $\Psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors
 $\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$
 $\Psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.5]
 $\Psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]
 $\Psi_{h,N} = (h/(2 \cdot h_{ef}))^{2/3} \leq 1.2$
 $\Psi_{h,N} = 0,95$ Coeff. related to the foundation height CEB [9.2.5]
 $\gamma_{M,sp} = 2,16$ Partial safety factor CEB [3.2.3.1]
 $F_{t,Rd,sp} = N_{Rk,c}^0 \cdot \Psi_{A,N} \cdot \Psi_{s,N} \cdot \Psi_{ec,N} \cdot \Psi_{re,N} \cdot \Psi_{ucr,N} \cdot \Psi_{h,N} / \gamma_{M,sp}$
 $F_{t,Rd,sp} = 52,21$ [kN] Design anchor resistance to splitting of concrete CEB [9.2.5]

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$
 $F_{t,Rd} = 32,08$ [kN] Tensile resistance of an anchor

BENDING OF THE BASE PLATE**Bending moment $M_{j,Ed,y}$**

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 780,49$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 341,84$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 96,23$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,y} = 96,23$ [kN] Tension resistance of a plate [6.2.4]

Bending moment $M_{j,Ed,z}$

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 528,33$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 232,42$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 64,15$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,z} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,z} = 64,15$ [kN] Tension resistance of a plate [6.2.4]

TENSILE RESISTANCE OF A COLUMN WEB**Bending moment $M_{j,Ed,y}$**

$t_{wc} = 5$ [mm] Effective thickness of the column web [6.2.6.3.(8)]

$b_{eff,t,wc} = 225$ [mm] Effective width of the web for tension [6.2.6.3.(2)]

$A_{vc} = 12,55$ [cm²] Shear area EN1993-1-1:[6.2.6.(3)]

$\omega = 0,44$ Reduction factor for interaction with shear [6.2.6.3.(4)]

$F_{t,wc,Rd,y} = \omega b_{eff,t,wc} t_{wc} f_{yc} / \gamma_{M0}$

$F_{t,wc,Rd,y} = 271,90$ [kN] Column web resistance [6.2.6.3.(1)]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$F_{T,Rd,y} = \min(F_{t,pl,Rd,y}, F_{t,wc,Rd,y})$

$F_{T,Rd,y} = 96,23$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

$F_{T,Rd,z} = F_{t,pl,Rd,z}$

$F_{T,Rd,z} = 64,15$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

CONNECTION CAPACITY CHECK

$$N_{j,Ed} / N_{j,Rd} \leq 1,0 \text{ (6.24)} \quad 0,00 < 1,00 \quad \text{verified} \quad (0,00)$$

$$e_y = 60544 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 84 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 27,33 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \text{ (6.23)} \quad 0,39 < 1,00 \quad \text{verified} \quad (0,39)$$

$$e_z = 48229 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,z} = 65 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,z} \quad [6.2.8.1.(2)]$$

$$z_{t,z} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,z} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,z} = 17,02 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \text{ (6.23)} \quad 0,50 < 1,00 \quad \text{verified} \quad (0,50)$$

$$M_{j,Ed,y} / M_{j,Rd,y} + M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \quad 0,89 < 1,00 \quad \text{verified} \quad (0,89)$$

SHEAR**BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE****Shear force $V_{j,Ed,y}$**

$$\alpha_{d,y} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,y} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,y} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,y} = k_{1,y} * \alpha_{b,y} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,y} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

Shear force $V_{j,Ed,z}$

$$\alpha_{d,z} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,z} = k_{1,z} * \alpha_{b,z} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,z} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

$\alpha_b = 0,25$ Coeff. for resistance calculation $F_{2,vb,Rd}$ [6.2.2.(7)]

$A_{vb} = 4,52 \text{ [cm}^2\text{]}$ Area of bolt section [6.2.2.(7)]

$f_{ub} = 800,00 \text{ [MPa]}$ Tensile strength of the anchor material [6.2.2.(7)]

$\gamma_{M2} = 1,25$ Partial safety factor [6.2.2.(7)]

$$F_{2,vb,Rd} = \alpha_b * f_{ub} * A_{vb} / \gamma_{M2}$$

$F_{2,vb,Rd} = 71,80 \text{ [kN]}$ Shear resistance of a bolt - without lever arm [6.2.2.(7)]

$\alpha_M = 2,00$ Factor related to the fastening of an anchor in the foundation CEB [9.3.2.2]

$M_{Rk,s} = 1,11 \text{ [kN*m]}$ Characteristic bending resistance of an anchor CEB [9.3.2.2]

$l_{sm} = 62 \text{ [mm]}$ Lever arm length CEB [9.3.2.2]

$\gamma_{Ms} = 1,20$ Partial safety factor CEB [3.2.3.2]

$$F_{v,Rd,sm} = \alpha_M * M_{Rk,s} / (l_{sm} * \gamma_{Ms})$$

$F_{v,Rd,sm} = 29,71 \text{ [kN]}$ Shear resistance of a bolt - with lever arm CEB [9.3.1]

CONCRETE PRY-OUT FAILURE

$N_{Rk,c} = 69,28 \text{ [kN]}$ Design uplift capacity CEB [9.2.4]

$k_3 = 2,00$ Factor related to the anchor length CEB [9.3.3]

$\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.1]

$$F_{v,Rd,cp} = k_3 * N_{Rk,c} / \gamma_{Mc}$$

$F_{v,Rd,cp} = 64,15 \text{ [kN]}$ Concrete resistance for pry-out failure CEB [9.3.1]

CONCRETE EDGE FAILURE

Shear force $V_{j,Ed,y}$

$V_{Rk,c,y}^0 = 70,02 \text{ [kN]}$ Characteristic resistance of an anchor

$\psi_{A,V,y} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,y} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,y} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,y} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,y} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,y} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$$F_{v,Rd,c,y} = V_{Rk,c,y}^0 * \psi_{A,V,y} * \psi_{h,V,y} * \psi_{s,V,y} * \psi_{ec,V,y} * \psi_{\alpha,V,y} * \psi_{ucr,V,y} / \gamma_{Mc}$$

$F_{v,Rd,c,y} = 19,45 \text{ [kN]}$ Concrete resistance for edge failure CEB [9.3.1]

Shear force $V_{j,Ed,z}$

$V_{Rk,c,z}^0 = 70,02$ [kN] Characteristic resistance of an anchor

$\psi_{A,V,z} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,z} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,z} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,z} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,z} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,z} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$F_{v,Rd,c,z} = V_{Rk,c,z}^0 \psi_{A,V,z} \psi_{h,V,z} \psi_{s,V,z} \psi_{ec,V,z} \psi_{\alpha,V,z} \psi_{ucr,V,z} / \gamma_{Mc}$

$F_{v,Rd,c,z} = 19,45$ [kN] Concrete resistance for edge failure CEB [9.3.1]

SPLITTING RESISTANCE

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete [6.2.2.(6)]

$N_{c,Ed} = 0,18$ [kN] Compressive force [6.2.2.(6)]

$F_{f,Rd} = C_{f,d} N_{c,Ed}$

$F_{f,Rd} = 0,05$ [kN] Slip resistance [6.2.2.(6)]

SHEAR CHECK

$V_{j,Rd,y} = n_b \min(F_{1,vb,Rd,y}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{f,Rd}$

$V_{j,Rd,y} = 116,76$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,y} / V_{j,Rd,y} \leq 1,0$ $0,03 < 1,00$ verified (0,03)

$V_{j,Rd,z} = n_b \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$

$V_{j,Rd,z} = 116,76$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,04 < 1,00$ verified (0,04)

$V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,07 < 1,00$ verified (0,07)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp} = 113,72$ [MPa] Normal stress in a weld [4.5.3.(7)]

$\tau_{\perp} = 113,72$ [MPa] Perpendicular tangent stress [4.5.3.(7)]

$\tau_{yII} = 2,63$ [MPa] Tangent stress parallel to $V_{j,Ed,y}$ [4.5.3.(7)]

$\tau_{zII} = 3,30$ [MPa] Tangent stress parallel to $V_{j,Ed,z}$ [4.5.3.(7)]

$\beta_W = 0,85$ Resistance-dependent coefficient [4.5.3.(7)]

$\sigma_{\perp} / (0,9 f_u / \gamma_{M2}) \leq 1,0$ (4.1) $0,37 < 1,00$ verified (0,37)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{yII}^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \gamma_{M2}))} \leq 1,0$ (4.1) $0,56 < 1,00$ verified (0,56)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{zII}^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \gamma_{M2}))} \leq 1,0$ (4.1) $0,52 < 1,00$ verified (0,52)

CONNECTION STIFFNESS**Bending moment $M_{j,Ed,y}$**

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff}) / (1.275 \cdot E)}$$

$k_{13,y} = 27$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,y} = 6$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,y} = 1.6 \cdot A_b / L_b$$

$k_{16,y} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,y} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,y} = 23942,24$ [kN*m] Initial rotational stiffness [Table 6.12]

$S_{j,rig,y} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,y} \geq S_{j,rig,y}$ RIGID [5.2.2.5.(2)]

Bending moment $M_{j,Ed,z}$

$$k_{13,z} = E_c \cdot \sqrt{(A_{c,z}) / (1.275 \cdot E)}$$

$k_{13,z} = 24$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,z} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,z} = 2$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,z} = 1.6 \cdot A_b / L_b$$

$k_{16,z} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,z} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,z} = 13649,96$ [kN*m] Initial rotational stiffness [6.3.1.(4)]

$S_{j,rig,z} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,z} < S_{j,rig,z}$ SEMI-RIGID [5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

REMARKS

Horizontal distance bolt-plate edge is too small. 25 [mm] < 29 [mm]

Connection conforms to the code

Ratio 0,89



Autodesk Robot Structural Analysis Professional 2021

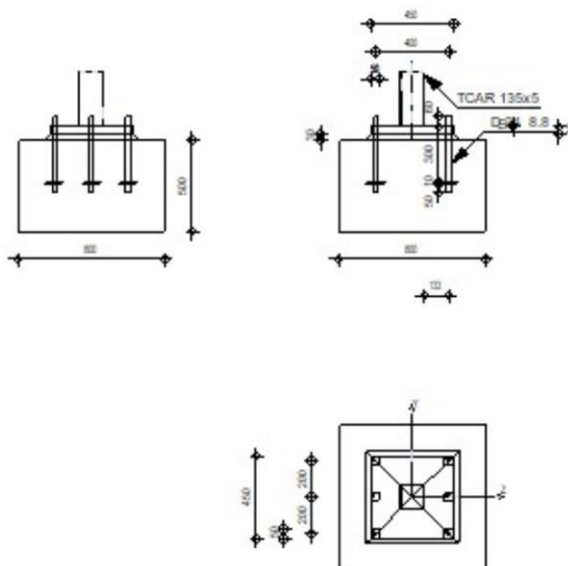
Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009 + CEB Design Guide:

Design of fastenings in concrete



Ratio
0,89



GENERAL

Connection no.: 54
 Connection name: Fixed column base
 Structure node: 130
 Structure bars: 6

GEOMETRY

COLUMN

Section: TCAR 135x5

Bar no.: 6

$L_c =$	2,40	[m]	Column length
$\alpha =$	0,0	[Deg]	Inclination angle
$h_c =$	135	[mm]	Height of column section
$b_{fc} =$	135	[mm]	Width of column section
$t_{wc} =$	5	[mm]	Thickness of the web of column section
$t_{fc} =$	5	[mm]	Thickness of the flange of column section
$r_c =$	11	[mm]	Radius of column section fillet
$A_c =$	25,10	[cm ²]	Cross-sectional area of a column
$I_{yc} =$	695,00	[cm ⁴]	Moment of inertia of the column section

Material: S275

 $f_{yc} = 275,00$ [MPa] Resistance $f_{uc} = 430,00$ [MPa] Yield strength of a material**COLUMN BASE** $l_{pd} = 450$ [mm] Length $b_{pd} = 450$ [mm] Width $t_{pd} = 40$ [mm] Thickness

Material: S275

 $f_{ypd} = 275,00$ [MPa] Resistance $f_{upd} = 430,00$ [MPa] Yield strength of a material**ANCHORAGE**

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 8.8 Anchor class

 $f_{yb} = 640,00$ [MPa] Yield strength of the anchor material $f_{ub} = 800,00$ [MPa] Tensile strength of the anchor material $d = 24$ [mm] Bolt diameter $A_s = 3,53$ [cm²] Effective section area of a bolt $A_v = 4,52$ [cm²] Area of bolt section $n_H = 2$ Number of bolt columns $n_V = 3$ Number of bolt rowsHorizontal spacing $e_{Hi} = 400$ [mm]Vertical spacing $e_{Vi} = 200$ [mm]**Anchor dimensions** $L_1 = 60$ [mm] $L_2 = 300$ [mm] $L_3 = 50$ [mm]**Anchor plate** $l_p = 100$ [mm] Length $b_p = 100$ [mm] Width $t_p = 10$ [mm] Thickness

Material: S275

 $f_y = 275,00$ [MPa] Resistance**Washer**

$l_{wd} = 50$ [mm] Length
 $b_{wd} = 50$ [mm] Width
 $t_{wd} = 10$ [mm] Thickness

MATERIAL FACTORS

$\gamma_{M0} = 1,00$ Partial safety factor
 $\gamma_{M2} = 1,25$ Partial safety factor
 $\gamma_C = 1,50$ Partial safety factor

SPREAD FOOTING

$L = 800$ [mm] Spread footing length
 $B = 800$ [mm] Spread footing width
 $H = 500$ [mm] Spread footing height

Concrete

Class C25/30

$f_{ck} = 25,00$ [MPa] Characteristic resistance for compression

Grout layer

$t_g = 30$ [mm] Thickness of leveling layer (grout)

$f_{ck,g} = 12,00$ [MPa] Characteristic resistance for compression

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete

WELDS

$a_p = 5$ [mm] Footing plate of the column base

LOADS

Case: 18: 1 * X -1 * Y SQRT(11*1.00;12*-1.00)

$N_{j,Ed} = -0,18$ [kN] Axial force
 $V_{j,Ed,y} = 3,55$ [kN] Shear force
 $V_{j,Ed,z} = 4,46$ [kN] Shear force
 $M_{j,Ed,y} = 10,70$ [kN*m] Bending moment
 $M_{j,Ed,z} = 8,52$ [kN*m] Bending moment

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

$f_{cd} = 16,67$ [MPa] Design compressive resistance EN 1992-1-1:3.1.6.(1)

$f_j = 19,75$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

$$c = t_p \sqrt{(f_{yp} / (3 \cdot f_j \cdot \gamma_{M0}))}$$

$c = 86$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$A_{c0} = 545,02$ [cm²] Area of the joint between the base plate and the foundation EN 1992-1-1:6.7.(3)

$A_{c1} = 4256,09$ [cm²] Maximum design area of load distribution EN 1992-1-1:6.7.(3)

$$F_{rd,u} = A_{c0} \cdot f_{cd} \cdot \sqrt{(A_{c1} / A_{c0})} \leq 3 \cdot A_{c0} \cdot f_{cd}$$

$F_{rd,u} = 2538,40$ [kN] Bearing resistance of concrete EN 1992-1-1:6.7.(3)

$\beta_j = 0,67$ Reduction factor for compression [6.2.5.(7)]

$$f_{jd} = \beta_j \cdot F_{rd,u} / (b_{eff} \cdot l_{eff})$$

$f_{jd} = 31,05$ [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n} = 877,03$ [cm²] Bearing area for compression [6.2.8.2.(1)]

$A_{c,y} = 438,51$ [cm²] Bearing area for bending My [6.2.8.3.(1)]

$A_{c,z} = 438,51$ [cm²] Bearing area for bending Mz [6.2.8.3.(1)]

$$F_{c,Rd,i} = A_{c,i} \cdot f_{jd}$$

$F_{c,Rd,n} = 2723,13$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]

$F_{c,Rd,y} = 1361,57$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

$F_{c,Rd,z} = 1361,57$ [kN] Bearing resistance of concrete for bending Mz [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL = 1,00$ Section class EN 1993-1-1:5.5.2

$W_{pl,y} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,y} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,y} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$$

$F_{c,fc,Rd,y} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

$W_{pl,z} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,z} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,z} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,z} = M_{c,Rd,z} / h_{f,z}$$

$F_{c,fc,Rd,z} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$$N_{j,Rd} = F_{c,Rd,n}$$

$N_{j,Rd} = 2723,13$ [kN] Resistance of a spread footing for axial compression [6.2.8.2.(1)]

$$F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$$

$$F_{C,Rd,y} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

$$F_{C,Rd,z} = \min(F_{c,Rd,z}, F_{c,fc,Rd,z})$$

$$F_{C,Rd,z} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

TENSION ZONE

STEEL FAILURE

$$A_b = 3,53 \text{ [cm}^2\text{]} \quad \text{Effective anchor area} \quad [\text{Table 3.4}]$$

$$f_{ub} = 800,00 \text{ [MPa]} \quad \text{Tensile strength of the anchor material} \quad [\text{Table 3.4}]$$

$$\text{Beta} = 0,85 \quad \text{Reduction factor of anchor resistance} \quad [3.6.1.(3)]$$

$$F_{t,Rd,s1} = \text{beta} \cdot 0.9 \cdot f_{ub} \cdot A_b / \gamma_{M2}$$

$$F_{t,Rd,s1} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad [\text{Table 3.4}]$$

$$\gamma_{Ms} = 1,20 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.2]}$$

$$f_{yb} = 640,00 \text{ [MPa]} \quad \text{Yield strength of the anchor material} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s2} = f_{yb} \cdot A_b / \gamma_{Ms}$$

$$F_{t,Rd,s2} = 188,27 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$$

$$F_{t,Rd,s} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure}$$

PULL-OUT FAILURE

$$f_{ck} = 25,00 \text{ [MPa]} \quad \text{Characteristic compressive strength of concrete} \quad \text{EN 1992-1:[3.1.2]}$$

$$A_h = 95,48 \text{ [cm}^2\text{]} \quad \text{Bearing area of the head} \quad \text{CEB [15.1.2.3]}$$

$$p_k = 175,00 \text{ [MPa]} \quad \text{Characteristic strength of concrete (pull-out)} \quad \text{CEB [15.1.2.3]}$$

$$\gamma_{Mp} = 2,16 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.1]}$$

$$F_{t,Rd,p} = p_k \cdot A_h / \gamma_{Mp}$$

$$F_{t,Rd,p} = 828,79 \text{ [kN]} \quad \text{Design uplift capacity} \quad \text{CEB [9.2.3]}$$

CONCRETE CONE FAILURE

$$h_{ef} = 133 \text{ [mm]} \quad \text{Effective anchorage depth} \quad \text{CEB [9.2.4]}$$

$$N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$$

$$N_{Rk,c}^0 = 69,28 \text{ [kN]} \quad \text{Characteristic resistance of an anchor} \quad \text{CEB [9.2.4]}$$

$$s_{cr,N} = 400 \text{ [mm]} \quad \text{Critical width of the concrete cone} \quad \text{CEB [9.2.4]}$$

$$c_{cr,N} = 200 \text{ [mm]} \quad \text{Critical edge distance} \quad \text{CEB [9.2.4]}$$

$$A_{c,N0} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Maximum area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$A_{c,N} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Actual area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$\psi_{A,N} = A_{c,N} / A_{c,N0}$$

$\Psi_{A,N} = 1,00$ Factor related to anchor spacing and edge distance CEB [9.2.4]

$c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.4]

$\Psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$

$\Psi_{s,N} = 1,00$ Factor taking account the influence of edges of the concrete member on the distribution of stresses

$\Psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors

$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$

$\Psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.

$\Psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.

$\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.

$F_{t,Rd,c} = N_{Rk,c} \cdot \Psi_{A,N} \cdot \Psi_{s,N} \cdot \Psi_{ec,N} \cdot \Psi_{re,N} \cdot \Psi_{ucr,N} / \gamma_{Mc}$

$F_{t,Rd,c} = 32,08$ [kN] Design anchor resistance to concrete cone failure EN 1992-1:[8.4.2.(2)]

SPLITTING FAILURE

$h_{ef} = 270$ [mm] Effective anchorage depth CEB [9.2.5]

$N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$

$N_{Rk,c}^0 = 199,64$ [kN] Design uplift capacity CEB [9.2.5]

$s_{cr,N} = 540$ [mm] Critical width of the concrete cone CEB [9.2.5]

$c_{cr,N} = 270$ [mm] Critical edge distance CEB [9.2.5]

$A_{c,N0} = 2916,00$ [cm²] Maximum area of concrete cone CEB [9.2.5]

$A_{c,N} = 1880,00$ [cm²] Actual area of concrete cone CEB [9.2.5]

$\Psi_{A,N} = A_{c,N}/A_{c,N0}$

$\Psi_{A,N} = 0,64$ Factor related to anchor spacing and edge distance CEB [9.2.5]

$c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.5]

$\Psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$

$\Psi_{s,N} = 0,92$ Factor taking account the influence of edges of the concrete member on the distribution of stresses

$\Psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors

$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$

$\Psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.5]

$\Psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]

$\Psi_{h,N} = (h/(2 \cdot h_{ef}))^{2/3} \leq 1.2$

$\Psi_{h,N} = 0,95$ Coeff. related to the foundation height CEB [9.2.5]

$\gamma_{M,sp} = 2,16$ Partial safety factor CEB [3.2.3.1]

$F_{t,Rd,sp} = N_{Rk,c} \cdot \Psi_{A,N} \cdot \Psi_{s,N} \cdot \Psi_{ec,N} \cdot \Psi_{re,N} \cdot \Psi_{ucr,N} \cdot \Psi_{h,N} / \gamma_{M,sp}$

$F_{t,Rd,sp} = 52,21$ [kN] Design anchor resistance to splitting of concrete CEB [9.2.5]

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$

$F_{t,Rd} = 32,08$ [kN] Tensile resistance of an anchor

BENDING OF THE BASE PLATE**Bending moment $M_{j,Ed,y}$**

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 780,49$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 341,84$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 96,23$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,y} = 96,23$ [kN] Tension resistance of a plate [6.2.4]

Bending moment $M_{j,Ed,z}$

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 528,33$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 232,42$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 64,15$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,z} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,z} = 64,15$ [kN] Tension resistance of a plate [6.2.4]

TENSILE RESISTANCE OF A COLUMN WEB**Bending moment $M_{j,Ed,y}$**

$t_{wc} = 5$ [mm] Effective thickness of the column web [6.2.6.3.(8)]

$b_{eff,t,wc} = 225$ [mm] Effective width of the web for tension [6.2.6.3.(2)]

$A_{vc} = 12,55$ [cm²] Shear area EN1993-1-1:[6.2.6.(3)]

$\omega = 0,44$ Reduction factor for interaction with shear [6.2.6.3.(4)]

$F_{t,wc,Rd,y} = \omega b_{eff,t,wc} t_{wc} f_{yc} / \gamma_{M0}$

$F_{t,wc,Rd,y} = 271,90$ [kN] Column web resistance [6.2.6.3.(1)]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$F_{T,Rd,y} = \min(F_{t,pl,Rd,y}, F_{t,wc,Rd,y})$

$F_{T,Rd,y} = 96,23$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

$F_{T,Rd,z} = F_{t,pl,Rd,z}$

$F_{T,Rd,z} = 64,15$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

CONNECTION CAPACITY CHECK

$$N_{j,Ed} / N_{j,Rd} \leq 1,0 \text{ (6.24)} \quad 0,00 < 1,00 \quad \text{verified} \quad (0,00)$$

$$e_y = 60825 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 84 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 27,33 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \text{ (6.23)} \quad 0,39 < 1,00 \quad \text{verified} \quad (0,39)$$

$$e_z = 48453 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,z} = 65 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,z} \quad [6.2.8.1.(2)]$$

$$z_{t,z} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,z} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,z} = 17,02 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \text{ (6.23)} \quad 0,50 < 1,00 \quad \text{verified} \quad (0,50)$$

$$M_{j,Ed,y} / M_{j,Rd,y} + M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \quad 0,89 < 1,00 \quad \text{verified} \quad (0,89)$$

SHEAR**BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE****Shear force $V_{j,Ed,y}$**

$$\alpha_{d,y} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,y} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,y} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,y} = k_{1,y} * \alpha_{b,y} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,y} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

Shear force $V_{j,Ed,z}$

$$\alpha_{d,z} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,z} = k_{1,z} * \alpha_{b,z} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,z} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

$\alpha_b = 0,25$ Coeff. for resistance calculation $F_{2,vb,Rd}$ [6.2.2.(7)]

$A_{vb} = 4,52 \text{ [cm}^2\text{]}$ Area of bolt section [6.2.2.(7)]

$f_{ub} = 800,00 \text{ [MPa]}$ Tensile strength of the anchor material [6.2.2.(7)]

$\gamma_{M2} = 1,25$ Partial safety factor [6.2.2.(7)]

$$F_{2,vb,Rd} = \alpha_b * f_{ub} * A_{vb} / \gamma_{M2}$$

$F_{2,vb,Rd} = 71,80 \text{ [kN]}$ Shear resistance of a bolt - without lever arm [6.2.2.(7)]

$\alpha_M = 2,00$ Factor related to the fastening of an anchor in the foundation CEB [9.3.2.2]

$M_{Rk,s} = 1,11 \text{ [kN*m]}$ Characteristic bending resistance of an anchor CEB [9.3.2.2]

$l_{sm} = 62 \text{ [mm]}$ Lever arm length CEB [9.3.2.2]

$\gamma_{Ms} = 1,20$ Partial safety factor CEB [3.2.3.2]

$$F_{v,Rd,sm} = \alpha_M * M_{Rk,s} / (l_{sm} * \gamma_{Ms})$$

$F_{v,Rd,sm} = 29,71 \text{ [kN]}$ Shear resistance of a bolt - with lever arm CEB [9.3.1]

CONCRETE PRY-OUT FAILURE

$N_{Rk,c} = 69,28 \text{ [kN]}$ Design uplift capacity CEB [9.2.4]

$k_3 = 2,00$ Factor related to the anchor length CEB [9.3.3]

$\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.1]

$$F_{v,Rd,cp} = k_3 * N_{Rk,c} / \gamma_{Mc}$$

$F_{v,Rd,cp} = 64,15 \text{ [kN]}$ Concrete resistance for pry-out failure CEB [9.3.1]

CONCRETE EDGE FAILURE

Shear force $V_{j,Ed,y}$

$V_{Rk,c,y}^0 = 70,02 \text{ [kN]}$ Characteristic resistance of an anchor

$\psi_{A,V,y} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,y} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,y} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,y} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,y} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,y} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$$F_{v,Rd,c,y} = V_{Rk,c,y}^0 * \psi_{A,V,y} * \psi_{h,V,y} * \psi_{s,V,y} * \psi_{ec,V,y} * \psi_{\alpha,V,y} * \psi_{ucr,V,y} / \gamma_{Mc}$$

$F_{v,Rd,c,y} = 19,45 \text{ [kN]}$ Concrete resistance for edge failure CEB [9.3.1]

Shear force $V_{j,Ed,z}$

$V_{Rk,c,z}^0 = 70,02$ [kN] Characteristic resistance of an anchor

$\psi_{A,V,z} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,z} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,z} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,z} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,z} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,z} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$F_{v,Rd,c,z} = V_{Rk,c,z}^0 \cdot \psi_{A,V,z} \cdot \psi_{h,V,z} \cdot \psi_{s,V,z} \cdot \psi_{ec,V,z} \cdot \psi_{\alpha,V,z} \cdot \psi_{ucr,V,z} / \gamma_{Mc}$

$F_{v,Rd,c,z} = 19,45$ [kN] Concrete resistance for edge failure CEB [9.3.1]

SPLITTING RESISTANCE

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete [6.2.2.(6)]

$N_{c,Ed} = 0,18$ [kN] Compressive force [6.2.2.(6)]

$F_{f,Rd} = C_{f,d} \cdot N_{c,Ed}$

$F_{f,Rd} = 0,05$ [kN] Slip resistance [6.2.2.(6)]

SHEAR CHECK

$V_{j,Rd,y} = n_b \cdot \min(F_{1,vb,Rd,y}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{f,Rd}$

$V_{j,Rd,y} = 116,76$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,y} / V_{j,Rd,y} \leq 1,0$ $0,03 < 1,00$ verified (0,03)

$V_{j,Rd,z} = n_b \cdot \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$

$V_{j,Rd,z} = 116,76$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,04 < 1,00$ verified (0,04)

$V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,07 < 1,00$ verified (0,07)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp} = 113,72$ [MPa] Normal stress in a weld [4.5.3.(7)]

$\tau_{\perp} = 113,72$ [MPa] Perpendicular tangent stress [4.5.3.(7)]

$\tau_{yII} = 2,63$ [MPa] Tangent stress parallel to $V_{j,Ed,y}$ [4.5.3.(7)]

$\tau_{zII} = 3,30$ [MPa] Tangent stress parallel to $V_{j,Ed,z}$ [4.5.3.(7)]

$\beta_W = 0,85$ Resistance-dependent coefficient [4.5.3.(7)]

$\sigma_{\perp} / (0,9 \cdot f_u / \gamma_{M2}) \leq 1,0$ (4.1) $0,37 < 1,00$ verified (0,37)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{yII}^2 + \tau_{zII}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1,0$ (4.1) $0,56 < 1,00$ verified (0,56)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{zII}^2 + \tau_{yII}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1,0$ (4.1) $0,52 < 1,00$ verified (0,52)

CONNECTION STIFFNESS**Bending moment $M_{j,Ed,y}$**

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff})} / (1.275 \cdot E)$$

$k_{13,y} = 27$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,y} = 6$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,y} = 1.6 \cdot A_b / L_b$$

$k_{16,y} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,y} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,y} = 23942,12$ [kN*m] Initial rotational stiffness [Table 6.12]

$S_{j,rig,y} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,y} \geq S_{j,rig,y}$ RIGID [5.2.2.5.(2)]

Bending moment $M_{j,Ed,z}$

$$k_{13,z} = E_c \cdot \sqrt{(A_{c,z})} / (1.275 \cdot E)$$

$k_{13,z} = 24$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,z} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,z} = 2$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,z} = 1.6 \cdot A_b / L_b$$

$k_{16,z} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,z} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,z} = 13649,89$ [kN*m] Initial rotational stiffness [6.3.1.(4)]

$S_{j,rig,z} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,z} < S_{j,rig,z}$ SEMI-RIGID [5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

REMARKS

Horizontal distance bolt-plate edge is too small. 25 [mm] < 29 [mm]

Connection conforms to the code

Ratio 0,89

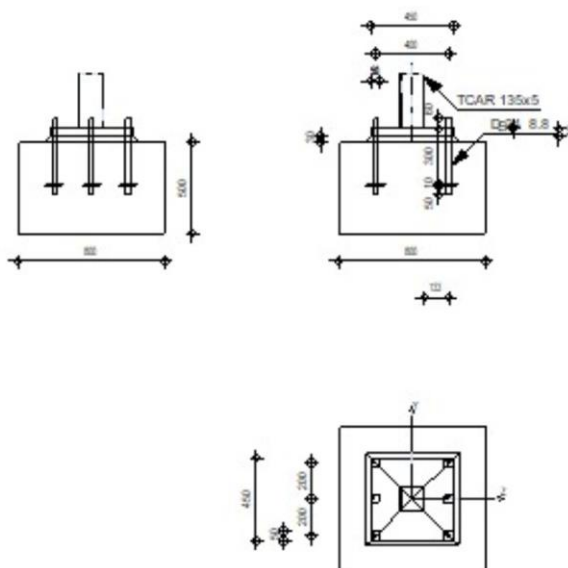


Autodesk Robot Structural Analysis Professional 2021

Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009 + CEB Design Guide:

Design of fastenings in concrete

Ratio
0,95**GENERAL**

Connection no.: 55

Connection name: Fixed column base

Structure node: 5

Structure bars: 3

GEOMETRY**COLUMN**

Section: TCAR 135x5

Bar no.: 3

$L_c =$	2,40	[m]	Column length
$\alpha =$	0,0	[Deg]	Inclination angle
$h_c =$	135	[mm]	Height of column section
$b_{fc} =$	135	[mm]	Width of column section
$t_{wc} =$	5	[mm]	Thickness of the web of column section
$t_{fc} =$	5	[mm]	Thickness of the flange of column section
$r_c =$	11	[mm]	Radius of column section fillet
$A_c =$	25,10	[cm ²]	Cross-sectional area of a column
$I_{yc} =$	695,00	[cm ⁴]	Moment of inertia of the column section

Material: S275

 $f_{yc} = 275,00$ [MPa] Resistance $f_{uc} = 430,00$ [MPa] Yield strength of a material**COLUMN BASE** $l_{pd} = 450$ [mm] Length $b_{pd} = 450$ [mm] Width $t_{pd} = 40$ [mm] Thickness

Material: S275

 $f_{ypd} = 275,00$ [MPa] Resistance $f_{upd} = 430,00$ [MPa] Yield strength of a material**ANCHORAGE**

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 8.8 Anchor class

 $f_{yb} = 640,00$ [MPa] Yield strength of the anchor material $f_{ub} = 800,00$ [MPa] Tensile strength of the anchor material $d = 24$ [mm] Bolt diameter $A_s = 3,53$ [cm²] Effective section area of a bolt $A_v = 4,52$ [cm²] Area of bolt section $n_H = 2$ Number of bolt columns $n_V = 3$ Number of bolt rowsHorizontal spacing $e_{Hi} = 400$ [mm]Vertical spacing $e_{Vi} = 200$ [mm]**Anchor dimensions** $L_1 = 60$ [mm] $L_2 = 300$ [mm] $L_3 = 50$ [mm]**Anchor plate** $l_p = 100$ [mm] Length $b_p = 100$ [mm] Width $t_p = 10$ [mm] Thickness

Material: S275

 $f_y = 275,00$ [MPa] Resistance**Washer**

$l_{wd} = 50$ [mm] Length
 $b_{wd} = 50$ [mm] Width
 $t_{wd} = 10$ [mm] Thickness

MATERIAL FACTORS

$\gamma_{M0} = 1,00$ Partial safety factor
 $\gamma_{M2} = 1,25$ Partial safety factor
 $\gamma_C = 1,50$ Partial safety factor

SPREAD FOOTING

$L = 800$ [mm] Spread footing length
 $B = 800$ [mm] Spread footing width
 $H = 500$ [mm] Spread footing height

Concrete

Class C25/30

$f_{ck} = 25,00$ [MPa] Characteristic resistance for compression

Grout layer

$t_g = 30$ [mm] Thickness of leveling layer (grout)

$f_{ck,g} = 12,00$ [MPa] Characteristic resistance for compression

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete

WELDS

$a_p = 5$ [mm] Footing plate of the column base

LOADS

Case: 18: 1 * X -1 * Y SQRT(11*1.00;12*-1.00)

$N_{j,Ed} = -2,67$ [kN] Axial force
 $V_{j,Ed,y} = 6,70$ [kN] Shear force
 $V_{j,Ed,z} = 6,19$ [kN] Shear force
 $M_{j,Ed,y} = 8,98$ [kN*m] Bending moment
 $M_{j,Ed,z} = 10,80$ [kN*m] Bending moment

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

$f_{cd} = 16,67$ [MPa] Design compressive resistance EN 1992-1-1:3.1.6.(1)

$f_j = 19,75$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

$$c = t_p \sqrt{(f_{yp} / (3 \cdot f_j \cdot \gamma_{M0}))}$$

$c = 86$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$A_{c0} = 545,02$ [cm²] Area of the joint between the base plate and the foundation EN 1992-1-1:6.7.(3)

$A_{c1} = 4256,09$ [cm²] Maximum design area of load distribution EN 1992-1-1:6.7.(3)

$$F_{rd,u} = A_{c0} \cdot f_{cd} \cdot \sqrt{(A_{c1} / A_{c0})} \leq 3 \cdot A_{c0} \cdot f_{cd}$$

$F_{rd,u} = 2538,40$ [kN] Bearing resistance of concrete EN 1992-1-1:6.7.(3)

$\beta_j = 0,67$ Reduction factor for compression [6.2.5.(7)]

$$f_{jd} = \beta_j \cdot F_{rd,u} / (b_{eff} \cdot l_{eff})$$

$f_{jd} = 31,05$ [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n} = 877,03$ [cm²] Bearing area for compression [6.2.8.2.(1)]

$A_{c,y} = 438,51$ [cm²] Bearing area for bending My [6.2.8.3.(1)]

$A_{c,z} = 438,51$ [cm²] Bearing area for bending Mz [6.2.8.3.(1)]

$$F_{c,Rd,i} = A_{c,i} \cdot f_{jd}$$

$F_{c,Rd,n} = 2723,13$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]

$F_{c,Rd,y} = 1361,57$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

$F_{c,Rd,z} = 1361,57$ [kN] Bearing resistance of concrete for bending Mz [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL = 1,00$ Section class EN 1993-1-1:5.5.2

$W_{pl,y} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,y} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,y} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$$

$F_{c,fc,Rd,y} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

$W_{pl,z} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,z} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,z} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,z} = M_{c,Rd,z} / h_{f,z}$$

$F_{c,fc,Rd,z} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$$N_{j,Rd} = F_{c,Rd,n}$$

$N_{j,Rd} = 2723,13$ [kN] Resistance of a spread footing for axial compression [6.2.8.2.(1)]

$$F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$$

$$F_{C,Rd,y} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

$$F_{C,Rd,z} = \min(F_{c,Rd,z}, F_{c,fc,Rd,z})$$

$$F_{C,Rd,z} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

TENSION ZONE

STEEL FAILURE

$$A_b = 3,53 \text{ [cm}^2\text{]} \quad \text{Effective anchor area} \quad [\text{Table 3.4}]$$

$$f_{ub} = 800,00 \text{ [MPa]} \quad \text{Tensile strength of the anchor material} \quad [\text{Table 3.4}]$$

$$\text{Beta} = 0,85 \quad \text{Reduction factor of anchor resistance} \quad [3.6.1.(3)]$$

$$F_{t,Rd,s1} = \text{beta} \cdot 0.9 \cdot f_{ub} \cdot A_b / \gamma_{M2}$$

$$F_{t,Rd,s1} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad [\text{Table 3.4}]$$

$$\gamma_{Ms} = 1,20 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.2]}$$

$$f_{yb} = 640,00 \text{ [MPa]} \quad \text{Yield strength of the anchor material} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s2} = f_{yb} \cdot A_b / \gamma_{Ms}$$

$$F_{t,Rd,s2} = 188,27 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$$

$$F_{t,Rd,s} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure}$$

PULL-OUT FAILURE

$$f_{ck} = 25,00 \text{ [MPa]} \quad \text{Characteristic compressive strength of concrete} \quad \text{EN 1992-1:[3.1.2]}$$

$$A_h = 95,48 \text{ [cm}^2\text{]} \quad \text{Bearing area of the head} \quad \text{CEB [15.1.2.3]}$$

$$p_k = 175,00 \text{ [MPa]} \quad \text{Characteristic strength of concrete (pull-out)} \quad \text{CEB [15.1.2.3]}$$

$$\gamma_{Mp} = 2,16 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.1]}$$

$$F_{t,Rd,p} = p_k \cdot A_h / \gamma_{Mp}$$

$$F_{t,Rd,p} = 828,79 \text{ [kN]} \quad \text{Design uplift capacity} \quad \text{CEB [9.2.3]}$$

CONCRETE CONE FAILURE

$$h_{ef} = 133 \text{ [mm]} \quad \text{Effective anchorage depth} \quad \text{CEB [9.2.4]}$$

$$N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$$

$$N_{Rk,c}^0 = 69,28 \text{ [kN]} \quad \text{Characteristic resistance of an anchor} \quad \text{CEB [9.2.4]}$$

$$s_{cr,N} = 400 \text{ [mm]} \quad \text{Critical width of the concrete cone} \quad \text{CEB [9.2.4]}$$

$$c_{cr,N} = 200 \text{ [mm]} \quad \text{Critical edge distance} \quad \text{CEB [9.2.4]}$$

$$A_{c,N0} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Maximum area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$A_{c,N} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Actual area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$\psi_{A,N} = A_{c,N} / A_{c,N0}$$

$\psi_{A,N} = 1,00$ Factor related to anchor spacing and edge distance CEB [9.2.4]
 $c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.4]
 $\psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$
 $\psi_{s,N} = 1,00$ Factor taking account the influence of edges of the concrete member on the distribution of stresses
 $\psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors
 $\psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$
 $\psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.5]
 $\psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]
 $\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.1]
 $F_{t,Rd,c} = N_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{ucr,N} / \gamma_{Mc}$
 $F_{t,Rd,c} = 32,08$ [kN] Design anchor resistance to concrete cone failure EN 1992-1-1:8.4.2.(2)

SPLITTING FAILURE

$h_{ef} = 270$ [mm] Effective anchorage depth CEB [9.2.5]
 $N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$
 $N_{Rk,c}^0 = 199,64$ [kN] Design uplift capacity CEB [9.2.5]
 $s_{cr,N} = 540$ [mm] Critical width of the concrete cone CEB [9.2.5]
 $c_{cr,N} = 270$ [mm] Critical edge distance CEB [9.2.5]
 $A_{c,N0} = 2916,00$ [cm²] Maximum area of concrete cone CEB [9.2.5]
 $A_{c,N} = 1880,00$ [cm²] Actual area of concrete cone CEB [9.2.5]
 $\psi_{A,N} = A_{c,N}/A_{c,N0}$
 $\psi_{A,N} = 0,64$ Factor related to anchor spacing and edge distance CEB [9.2.5]
 $c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.5]
 $\psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$
 $\psi_{s,N} = 0,92$ Factor taking account the influence of edges of the concrete member on the distribution of stresses
 $\psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors
 $\psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$
 $\psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.5]
 $\psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]
 $\psi_{h,N} = (h/(2 \cdot h_{ef}))^{2/3} \leq 1.2$
 $\psi_{h,N} = 0,95$ Coeff. related to the foundation height CEB [9.2.5]
 $\gamma_{M,sp} = 2,16$ Partial safety factor CEB [3.2.3.1]
 $F_{t,Rd,sp} = N_{Rk,c}^0 \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{ucr,N} \cdot \psi_{h,N} / \gamma_{M,sp}$
 $F_{t,Rd,sp} = 52,21$ [kN] Design anchor resistance to splitting of concrete CEB [9.2.5]

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$
 $F_{t,Rd} = 32,08$ [kN] Tensile resistance of an anchor

BENDING OF THE BASE PLATE**Bending moment $M_{j,Ed,y}$**

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 780,49$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 341,84$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 96,23$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,y} = 96,23$ [kN] Tension resistance of a plate [6.2.4]

Bending moment $M_{j,Ed,z}$

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 528,33$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 232,42$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 64,15$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,z} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,z} = 64,15$ [kN] Tension resistance of a plate [6.2.4]

TENSILE RESISTANCE OF A COLUMN WEB**Bending moment $M_{j,Ed,y}$**

$t_{wc} = 5$ [mm] Effective thickness of the column web [6.2.6.3.(8)]

$b_{eff,t,wc} = 225$ [mm] Effective width of the web for tension [6.2.6.3.(2)]

$A_{vc} = 12,55$ [cm²] Shear area EN1993-1-1:[6.2.6.(3)]

$\omega = 0,44$ Reduction factor for interaction with shear [6.2.6.3.(4)]

$F_{t,wc,Rd,y} = \omega b_{eff,t,wc} t_{wc} f_{yc} / \gamma_{M0}$

$F_{t,wc,Rd,y} = 271,90$ [kN] Column web resistance [6.2.6.3.(1)]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$F_{T,Rd,y} = \min(F_{t,pl,Rd,y}, F_{t,wc,Rd,y})$

$F_{T,Rd,y} = 96,23$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

$F_{T,Rd,z} = F_{t,pl,Rd,z}$

$F_{T,Rd,z} = 64,15$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

CONNECTION CAPACITY CHECK

$$N_{j,Ed} / N_{j,Rd} \leq 1,0 \text{ (6.24)} \quad 0,00 < 1,00 \quad \text{verified} \quad (0,00)$$

$$e_y = 3366 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 84 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 27,99 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \text{ (6.23)} \quad 0,32 < 1,00 \quad \text{verified} \quad (0,32)$$

$$e_z = 4047 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,z} = 65 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,z} \quad [6.2.8.1.(2)]$$

$$z_{t,z} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,z} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,z} = 17,28 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \text{ (6.23)} \quad 0,63 < 1,00 \quad \text{verified} \quad (0,63)$$

$$M_{j,Ed,y} / M_{j,Rd,y} + M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \quad 0,95 < 1,00 \quad \text{verified} \quad (0,95)$$

SHEAR**BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE****Shear force $V_{j,Ed,y}$**

$$\alpha_{d,y} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,y} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,y} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,y} = k_{1,y} * \alpha_{b,y} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,y} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

Shear force $V_{j,Ed,z}$

$$\alpha_{d,z} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,z} = k_{1,z} * \alpha_{b,z} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,z} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

$\alpha_b = 0,25$ Coeff. for resistance calculation $F_{2,vb,Rd}$ [6.2.2.(7)]

$A_{vb} = 4,52 \text{ [cm}^2\text{]}$ Area of bolt section [6.2.2.(7)]

$f_{ub} = 800,00 \text{ [MPa]}$ Tensile strength of the anchor material [6.2.2.(7)]

$\gamma_{M2} = 1,25$ Partial safety factor [6.2.2.(7)]

$$F_{2,vb,Rd} = \alpha_b * f_{ub} * A_{vb} / \gamma_{M2}$$

$F_{2,vb,Rd} = 71,80 \text{ [kN]}$ Shear resistance of a bolt - without lever arm [6.2.2.(7)]

$\alpha_M = 2,00$ Factor related to the fastening of an anchor in the foundation CEB [9.3.2.2]

$M_{Rk,s} = 1,10 \text{ [kN*m]}$ Characteristic bending resistance of an anchor CEB [9.3.2.2]

$l_{sm} = 62 \text{ [mm]}$ Lever arm length CEB [9.3.2.2]

$\gamma_{Ms} = 1,20$ Partial safety factor CEB [3.2.3.2]

$$F_{v,Rd,sm} = \alpha_M * M_{Rk,s} / (l_{sm} * \gamma_{Ms})$$

$F_{v,Rd,sm} = 29,46 \text{ [kN]}$ Shear resistance of a bolt - with lever arm CEB [9.3.1]

CONCRETE PRY-OUT FAILURE

$N_{Rk,c} = 69,28 \text{ [kN]}$ Design uplift capacity CEB [9.2.4]

$k_3 = 2,00$ Factor related to the anchor length CEB [9.3.3]

$\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.1]

$$F_{v,Rd,cp} = k_3 * N_{Rk,c} / \gamma_{Mc}$$

$F_{v,Rd,cp} = 64,15 \text{ [kN]}$ Concrete resistance for pry-out failure CEB [9.3.1]

CONCRETE EDGE FAILURE

Shear force $V_{j,Ed,y}$

$V_{Rk,c,y}^0 = 70,02 \text{ [kN]}$ Characteristic resistance of an anchor

$\psi_{A,V,y} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,y} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,y} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,y} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,y} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,y} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$$F_{v,Rd,c,y} = V_{Rk,c,y}^0 * \psi_{A,V,y} * \psi_{h,V,y} * \psi_{s,V,y} * \psi_{ec,V,y} * \psi_{\alpha,V,y} * \psi_{ucr,V,y} / \gamma_{Mc}$$

$F_{v,Rd,c,y} = 19,45 \text{ [kN]}$ Concrete resistance for edge failure CEB [9.3.1]

Shear force $V_{j,Ed,z}$

$V_{Rk,c,z}^0 = 70,02$ [kN] Characteristic resistance of an anchor

$\psi_{A,V,z} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,z} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,z} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,z} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,z} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,z} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$F_{v,Rd,c,z} = V_{Rk,c,z}^0 \cdot \psi_{A,V,z} \cdot \psi_{h,V,z} \cdot \psi_{s,V,z} \cdot \psi_{ec,V,z} \cdot \psi_{\alpha,V,z} \cdot \psi_{ucr,V,z} / \gamma_{Mc}$

$F_{v,Rd,c,z} = 19,45$ [kN] Concrete resistance for edge failure CEB [9.3.1]

SPLITTING RESISTANCE

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete [6.2.2.(6)]

$N_{c,Ed} = 2,67$ [kN] Compressive force [6.2.2.(6)]

$F_{f,Rd} = C_{f,d} \cdot N_{c,Ed}$

$F_{f,Rd} = 0,80$ [kN] Slip resistance [6.2.2.(6)]

SHEAR CHECK

$V_{j,Rd,y} = n_b \cdot \min(F_{1,vb,Rd,y}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{f,Rd}$

$V_{j,Rd,y} = 117,51$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,y} / V_{j,Rd,y} \leq 1,0$ $0,06 < 1,00$ verified (0,06)

$V_{j,Rd,z} = n_b \cdot \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$

$V_{j,Rd,z} = 117,51$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,05 < 1,00$ verified (0,05)

$V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,11 < 1,00$ verified (0,11)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp} = 117,55$ [MPa] Normal stress in a weld [4.5.3.(7)]

$\tau_{\perp} = 117,55$ [MPa] Perpendicular tangent stress [4.5.3.(7)]

$\tau_{yII} = 4,96$ [MPa] Tangent stress parallel to $V_{j,Ed,y}$ [4.5.3.(7)]

$\tau_{zII} = 4,59$ [MPa] Tangent stress parallel to $V_{j,Ed,z}$ [4.5.3.(7)]

$\beta_W = 0,85$ Resistance-dependent coefficient [4.5.3.(7)]

$\sigma_{\perp} / (0,9 \cdot f_u / \gamma_{M2}) \leq 1,0$ (4.1) $0,38 < 1,00$ verified (0,38)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{yII}^2 + \tau_{zII}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1,0$ (4.1) $0,58 < 1,00$ verified (0,58)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{zII}^2 + \tau_{yII}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1,0$ (4.1) $0,55 < 1,00$ verified (0,55)

CONNECTION STIFFNESS**Bending moment $M_{j,Ed,y}$**

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff}) / (1.275 \cdot E)}$$

$k_{13,y} = 27$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,y} = 6$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,y} = 1.6 \cdot A_b / L_b$$

$k_{16,y} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,y} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,y} = 24414,18$ [kN*m] Initial rotational stiffness [Table 6.12]

$S_{j,rig,y} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,y} \geq S_{j,rig,y}$ RIGID [5.2.2.5.(2)]

Bending moment $M_{j,Ed,z}$

$$k_{13,z} = E_c \cdot \sqrt{(A_{c,z}) / (1.275 \cdot E)}$$

$k_{13,z} = 24$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,z} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,z} = 2$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,z} = 1.6 \cdot A_b / L_b$$

$k_{16,z} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,z} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,z} = 13821,89$ [kN*m] Initial rotational stiffness [6.3.1.(4)]

$S_{j,rig,z} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,z} < S_{j,rig,z}$ SEMI-RIGID [5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

REMARKS

Horizontal distance bolt-plate edge is too small. 25 [mm] < 29 [mm]

Connection conforms to the code

Ratio 0,95

Material: S275

 $f_{yc} = 275,00$ [MPa] Resistance $f_{uc} = 430,00$ [MPa] Yield strength of a material**COLUMN BASE** $l_{pd} = 450$ [mm] Length $b_{pd} = 450$ [mm] Width $t_{pd} = 40$ [mm] Thickness

Material: S275

 $f_{ypd} = 275,00$ [MPa] Resistance $f_{upd} = 430,00$ [MPa] Yield strength of a material**ANCHORAGE**

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 8.8 Anchor class

 $f_{yb} = 640,00$ [MPa] Yield strength of the anchor material $f_{ub} = 800,00$ [MPa] Tensile strength of the anchor material $d = 24$ [mm] Bolt diameter $A_s = 3,53$ [cm²] Effective section area of a bolt $A_v = 4,52$ [cm²] Area of bolt section $n_H = 2$ Number of bolt columns $n_V = 3$ Number of bolt rowsHorizontal spacing $e_{Hi} = 400$ [mm]Vertical spacing $e_{Vi} = 200$ [mm]**Anchor dimensions** $L_1 = 60$ [mm] $L_2 = 300$ [mm] $L_3 = 50$ [mm]**Anchor plate** $l_p = 100$ [mm] Length $b_p = 100$ [mm] Width $t_p = 10$ [mm] Thickness

Material: S275

 $f_y = 275,00$ [MPa] Resistance**Washer**

$l_{wd} = 50$ [mm] Length
 $b_{wd} = 50$ [mm] Width
 $t_{wd} = 10$ [mm] Thickness

MATERIAL FACTORS

$\gamma_{M0} = 1,00$ Partial safety factor
 $\gamma_{M2} = 1,25$ Partial safety factor
 $\gamma_C = 1,50$ Partial safety factor

SPREAD FOOTING

$L = 800$ [mm] Spread footing length
 $B = 800$ [mm] Spread footing width
 $H = 500$ [mm] Spread footing height

Concrete

Class C25/30

$f_{ck} = 25,00$ [MPa] Characteristic resistance for compression

Grout layer

$t_g = 30$ [mm] Thickness of leveling layer (grout)

$f_{ck,g} = 12,00$ [MPa] Characteristic resistance for compression

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete

WELDS

$a_p = 5$ [mm] Footing plate of the column base

LOADS

Case: 18: 1 * X -1 * Y SQRT(11*1.00;12*-1.00)

$N_{j,Ed} = -3,03$ [kN] Axial force
 $V_{j,Ed,y} = 6,73$ [kN] Shear force
 $V_{j,Ed,z} = 6,35$ [kN] Shear force
 $M_{j,Ed,y} = 9,12$ [kN*m] Bending moment
 $M_{j,Ed,z} = 10,83$ [kN*m] Bending moment

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

$f_{cd} = 16,67$ [MPa] Design compressive resistance EN 1992-1-1:3.1.6.(1)

$f_j = 19,75$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

$$c = t_p \sqrt{(f_{yp} / (3 \cdot f_j \cdot \gamma_{M0}))}$$

$c = 86$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$A_{c0} = 545,02$ [cm²] Area of the joint between the base plate and the foundation EN 1992-1-1:6.7.(3)

$A_{c1} = 4256,09$ [cm²] Maximum design area of load distribution EN 1992-1-1:6.7.(3)

$$F_{rd,u} = A_{c0} \cdot f_{cd} \cdot \sqrt{(A_{c1} / A_{c0})} \leq 3 \cdot A_{c0} \cdot f_{cd}$$

$F_{rd,u} = 2538,40$ [kN] Bearing resistance of concrete EN 1992-1-1:6.7.(3)

$\beta_j = 0,67$ Reduction factor for compression [6.2.5.(7)]

$$f_{jd} = \beta_j \cdot F_{rd,u} / (b_{eff} \cdot l_{eff})$$

$f_{jd} = 31,05$ [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n} = 877,03$ [cm²] Bearing area for compression [6.2.8.2.(1)]

$A_{c,y} = 438,51$ [cm²] Bearing area for bending My [6.2.8.3.(1)]

$A_{c,z} = 438,51$ [cm²] Bearing area for bending Mz [6.2.8.3.(1)]

$$F_{c,Rd,i} = A_{c,i} \cdot f_{jd}$$

$F_{c,Rd,n} = 2723,13$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]

$F_{c,Rd,y} = 1361,57$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

$F_{c,Rd,z} = 1361,57$ [kN] Bearing resistance of concrete for bending Mz [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL = 1,00$ Section class EN 1993-1-1:5.5.2

$W_{pl,y} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,y} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,y} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$$

$F_{c,fc,Rd,y} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

$W_{pl,z} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,z} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,z} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,z} = M_{c,Rd,z} / h_{f,z}$$

$F_{c,fc,Rd,z} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$$N_{j,Rd} = F_{c,Rd,n}$$

$N_{j,Rd} = 2723,13$ [kN] Resistance of a spread footing for axial compression [6.2.8.2.(1)]

$$F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$$

$$F_{C,Rd,y} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

$$F_{C,Rd,z} = \min(F_{c,Rd,z}, F_{c,fc,Rd,z})$$

$$F_{C,Rd,z} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

TENSION ZONE

STEEL FAILURE

$$A_b = 3,53 \text{ [cm}^2\text{]} \quad \text{Effective anchor area} \quad [\text{Table 3.4}]$$

$$f_{ub} = 800,00 \text{ [MPa]} \quad \text{Tensile strength of the anchor material} \quad [\text{Table 3.4}]$$

$$\text{Beta} = 0,85 \quad \text{Reduction factor of anchor resistance} \quad [3.6.1.(3)]$$

$$F_{t,Rd,s1} = \text{beta} \cdot 0.9 \cdot f_{ub} \cdot A_b / \gamma_{M2}$$

$$F_{t,Rd,s1} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad [\text{Table 3.4}]$$

$$\gamma_{Ms} = 1,20 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.2]}$$

$$f_{yb} = 640,00 \text{ [MPa]} \quad \text{Yield strength of the anchor material} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s2} = f_{yb} \cdot A_b / \gamma_{Ms}$$

$$F_{t,Rd,s2} = 188,27 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$$

$$F_{t,Rd,s} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure}$$

PULL-OUT FAILURE

$$f_{ck} = 25,00 \text{ [MPa]} \quad \text{Characteristic compressive strength of concrete} \quad \text{EN 1992-1:[3.1.2]}$$

$$A_h = 95,48 \text{ [cm}^2\text{]} \quad \text{Bearing area of the head} \quad \text{CEB [15.1.2.3]}$$

$$p_k = 175,00 \text{ [MPa]} \quad \text{Characteristic strength of concrete (pull-out)} \quad \text{CEB [15.1.2.3]}$$

$$\gamma_{Mp} = 2,16 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.1]}$$

$$F_{t,Rd,p} = p_k \cdot A_h / \gamma_{Mp}$$

$$F_{t,Rd,p} = 828,79 \text{ [kN]} \quad \text{Design uplift capacity} \quad \text{CEB [9.2.3]}$$

CONCRETE CONE FAILURE

$$h_{ef} = 133 \text{ [mm]} \quad \text{Effective anchorage depth} \quad \text{CEB [9.2.4]}$$

$$N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$$

$$N_{Rk,c}^0 = 69,28 \text{ [kN]} \quad \text{Characteristic resistance of an anchor} \quad \text{CEB [9.2.4]}$$

$$s_{cr,N} = 400 \text{ [mm]} \quad \text{Critical width of the concrete cone} \quad \text{CEB [9.2.4]}$$

$$c_{cr,N} = 200 \text{ [mm]} \quad \text{Critical edge distance} \quad \text{CEB [9.2.4]}$$

$$A_{c,N0} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Maximum area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$A_{c,N} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Actual area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$\psi_{A,N} = A_{c,N} / A_{c,N0}$$

$\Psi_{A,N} = 1,00$ Factor related to anchor spacing and edge distance CEB [9.2.4]

$c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.4]

$\Psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$

$\Psi_{s,N} = 1,00$ Factor taking account the influence of edges of the concrete member on the distribution of stresses

$\Psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors

$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$

$\Psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.

$\Psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.

$\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.

$F_{t,Rd,c} = N_{Rk,c} \cdot \Psi_{A,N} \cdot \Psi_{s,N} \cdot \Psi_{ec,N} \cdot \Psi_{re,N} \cdot \Psi_{ucr,N} / \gamma_{Mc}$

$F_{t,Rd,c} = 32,08$ [kN] Design anchor resistance to concrete cone failure EN 1992-1:[8.4.2.(2)]

SPLITTING FAILURE

$h_{ef} = 270$ [mm] Effective anchorage depth CEB [9.2.5]

$N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$

$N_{Rk,c}^0 = 199,64$ [kN] Design uplift capacity CEB [9.2.5]

$s_{cr,N} = 540$ [mm] Critical width of the concrete cone CEB [9.2.5]

$c_{cr,N} = 270$ [mm] Critical edge distance CEB [9.2.5]

$A_{c,N0} = 2916,00$ [cm²] Maximum area of concrete cone CEB [9.2.5]

$A_{c,N} = 1880,00$ [cm²] Actual area of concrete cone CEB [9.2.5]

$\Psi_{A,N} = A_{c,N}/A_{c,N0}$

$\Psi_{A,N} = 0,64$ Factor related to anchor spacing and edge distance CEB [9.2.5]

$c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.5]

$\Psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$

$\Psi_{s,N} = 0,92$ Factor taking account the influence of edges of the concrete member on the distribution of stresses

$\Psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors

$\Psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$

$\Psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.5]

$\Psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]

$\Psi_{h,N} = (h/(2 \cdot h_{ef}))^{2/3} \leq 1.2$

$\Psi_{h,N} = 0,95$ Coeff. related to the foundation height CEB [9.2.5]

$\gamma_{M,sp} = 2,16$ Partial safety factor CEB [3.2.3.1]

$F_{t,Rd,sp} = N_{Rk,c} \cdot \Psi_{A,N} \cdot \Psi_{s,N} \cdot \Psi_{ec,N} \cdot \Psi_{re,N} \cdot \Psi_{ucr,N} \cdot \Psi_{h,N} / \gamma_{M,sp}$

$F_{t,Rd,sp} = 52,21$ [kN] Design anchor resistance to splitting of concrete CEB [9.2.5]

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$

$F_{t,Rd} = 32,08$ [kN] Tensile resistance of an anchor

BENDING OF THE BASE PLATE**Bending moment $M_{j,Ed,y}$**

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 780,49$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 341,84$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 96,23$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,y} = 96,23$ [kN] Tension resistance of a plate [6.2.4]

Bending moment $M_{j,Ed,z}$

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 528,33$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 232,42$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 64,15$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,z} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,z} = 64,15$ [kN] Tension resistance of a plate [6.2.4]

TENSILE RESISTANCE OF A COLUMN WEB**Bending moment $M_{j,Ed,y}$**

$t_{wc} = 5$ [mm] Effective thickness of the column web [6.2.6.3.(8)]

$b_{eff,t,wc} = 225$ [mm] Effective width of the web for tension [6.2.6.3.(2)]

$A_{vc} = 12,55$ [cm²] Shear area EN1993-1-1:[6.2.6.(3)]

$\omega = 0,44$ Reduction factor for interaction with shear [6.2.6.3.(4)]

$F_{t,wc,Rd,y} = \omega b_{eff,t,wc} t_{wc} f_{yc} / \gamma_{M0}$

$F_{t,wc,Rd,y} = 271,90$ [kN] Column web resistance [6.2.6.3.(1)]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$F_{T,Rd,y} = \min(F_{t,pl,Rd,y}, F_{t,wc,Rd,y})$

$F_{T,Rd,y} = 96,23$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

$F_{T,Rd,z} = F_{t,pl,Rd,z}$

$F_{T,Rd,z} = 64,15$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

CONNECTION CAPACITY CHECK

$$N_{j,Ed} / N_{j,Rd} \leq 1,0 \text{ (6.24)} \quad 0,00 < 1,00 \quad \text{verified} \quad (0,00)$$

$$e_y = 3016 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 84 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 28,07 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \text{ (6.23)} \quad 0,32 < 1,00 \quad \text{verified} \quad (0,32)$$

$$e_z = 3579 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,z} = 65 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,z} \quad [6.2.8.1.(2)]$$

$$z_{t,z} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,z} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,z} = 17,31 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \text{ (6.23)} \quad 0,63 < 1,00 \quad \text{verified} \quad (0,63)$$

$$M_{j,Ed,y} / M_{j,Rd,y} + M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \quad 0,95 < 1,00 \quad \text{verified} \quad (0,95)$$

SHEAR**BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE****Shear force $V_{j,Ed,y}$**

$$\alpha_{d,y} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,y} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,y} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,y} = k_{1,y} * \alpha_{b,y} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,y} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

Shear force $V_{j,Ed,z}$

$$\alpha_{d,z} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,z} = k_{1,z} * \alpha_{b,z} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,z} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

$\alpha_b = 0,25$ Coeff. for resistance calculation $F_{2,vb,Rd}$ [6.2.2.(7)]

$A_{vb} = 4,52 \text{ [cm}^2\text{]}$ Area of bolt section [6.2.2.(7)]

$f_{ub} = 800,00 \text{ [MPa]}$ Tensile strength of the anchor material [6.2.2.(7)]

$\gamma_{M2} = 1,25$ Partial safety factor [6.2.2.(7)]

$$F_{2,vb,Rd} = \alpha_b * f_{ub} * A_{vb} / \gamma_{M2}$$

$F_{2,vb,Rd} = 71,80 \text{ [kN]}$ Shear resistance of a bolt - without lever arm [6.2.2.(7)]

$\alpha_M = 2,00$ Factor related to the fastening of an anchor in the foundation CEB [9.3.2.2]

$M_{Rk,s} = 1,10 \text{ [kN*m]}$ Characteristic bending resistance of an anchor CEB [9.3.2.2]

$l_{sm} = 62 \text{ [mm]}$ Lever arm length CEB [9.3.2.2]

$\gamma_{Ms} = 1,20$ Partial safety factor CEB [3.2.3.2]

$$F_{v,Rd,sm} = \alpha_M * M_{Rk,s} / (l_{sm} * \gamma_{Ms})$$

$F_{v,Rd,sm} = 29,45 \text{ [kN]}$ Shear resistance of a bolt - with lever arm CEB [9.3.1]

CONCRETE PRY-OUT FAILURE

$N_{Rk,c} = 69,28 \text{ [kN]}$ Design uplift capacity CEB [9.2.4]

$k_3 = 2,00$ Factor related to the anchor length CEB [9.3.3]

$\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.1]

$$F_{v,Rd,cp} = k_3 * N_{Rk,c} / \gamma_{Mc}$$

$F_{v,Rd,cp} = 64,15 \text{ [kN]}$ Concrete resistance for pry-out failure CEB [9.3.1]

CONCRETE EDGE FAILURE

Shear force $V_{j,Ed,y}$

$V_{Rk,c,y}^0 = 70,02 \text{ [kN]}$ Characteristic resistance of an anchor

$\psi_{A,V,y} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,y} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,y} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,y} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,y} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,y} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$$F_{v,Rd,c,y} = V_{Rk,c,y}^0 * \psi_{A,V,y} * \psi_{h,V,y} * \psi_{s,V,y} * \psi_{ec,V,y} * \psi_{\alpha,V,y} * \psi_{ucr,V,y} / \gamma_{Mc}$$

$F_{v,Rd,c,y} = 19,45 \text{ [kN]}$ Concrete resistance for edge failure CEB [9.3.1]

Shear force $V_{j,Ed,z}$

$V_{Rk,c,z}^0 = 70,02$ [kN] Characteristic resistance of an anchor

$\psi_{A,V,z} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,z} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,z} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,z} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,z} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,z} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$F_{v,Rd,c,z} = V_{Rk,c,z}^0 \cdot \psi_{A,V,z} \cdot \psi_{h,V,z} \cdot \psi_{s,V,z} \cdot \psi_{ec,V,z} \cdot \psi_{\alpha,V,z} \cdot \psi_{ucr,V,z} / \gamma_{Mc}$

$F_{v,Rd,c,z} = 19,45$ [kN] Concrete resistance for edge failure CEB [9.3.1]

SPLITTING RESISTANCE

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete [6.2.2.(6)]

$N_{c,Ed} = 3,03$ [kN] Compressive force [6.2.2.(6)]

$F_{f,Rd} = C_{f,d} \cdot N_{c,Ed}$

$F_{f,Rd} = 0,91$ [kN] Slip resistance [6.2.2.(6)]

SHEAR CHECK

$V_{j,Rd,y} = n_b \cdot \min(F_{1,vb,Rd,y}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{f,Rd}$

$V_{j,Rd,y} = 117,61$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,y} / V_{j,Rd,y} \leq 1,0$ $0,06 < 1,00$ verified (0,06)

$V_{j,Rd,z} = n_b \cdot \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$

$V_{j,Rd,z} = 117,61$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,05 < 1,00$ verified (0,05)

$V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,11 < 1,00$ verified (0,11)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp} = 118,58$ [MPa] Normal stress in a weld [4.5.3.(7)]

$\tau_{\perp} = 118,58$ [MPa] Perpendicular tangent stress [4.5.3.(7)]

$\tau_{yII} = 4,99$ [MPa] Tangent stress parallel to $V_{j,Ed,y}$ [4.5.3.(7)]

$\tau_{zII} = 4,70$ [MPa] Tangent stress parallel to $V_{j,Ed,z}$ [4.5.3.(7)]

$\beta_W = 0,85$ Resistance-dependent coefficient [4.5.3.(7)]

$\sigma_{\perp} / (0,9 \cdot f_u / \gamma_{M2}) \leq 1,0$ (4.1) $0,38 < 1,00$ verified (0,38)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{yII}^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1,0$ (4.1) $0,59 < 1,00$ verified (0,59)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{zII}^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1,0$ (4.1) $0,55 < 1,00$ verified (0,55)

CONNECTION STIFFNESS**Bending moment $M_{j,Ed,y}$**

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff}) / (1.275 \cdot E)}$$

$k_{13,y} = 27$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,y} = 6$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,y} = 1.6 \cdot A_b / L_b$$

$k_{16,y} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,y} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,y} = 24473,46$ [kN*m] Initial rotational stiffness [Table 6.12]

$S_{j,rig,y} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,y} \geq S_{j,rig,y}$ RIGID [5.2.2.5.(2)]

Bending moment $M_{j,Ed,z}$

$$k_{13,z} = E_c \cdot \sqrt{(A_{c,z}) / (1.275 \cdot E)}$$

$k_{13,z} = 24$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,z} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,z} = 2$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,z} = 1.6 \cdot A_b / L_b$$

$k_{16,z} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,z} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,z} = 13846,79$ [kN*m] Initial rotational stiffness [6.3.1.(4)]

$S_{j,rig,z} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,z} < S_{j,rig,z}$ SEMI-RIGID [5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

REMARKS

Horizontal distance bolt-plate edge is too small. 25 [mm] < 29 [mm]

Connection conforms to the code

Ratio 0,95



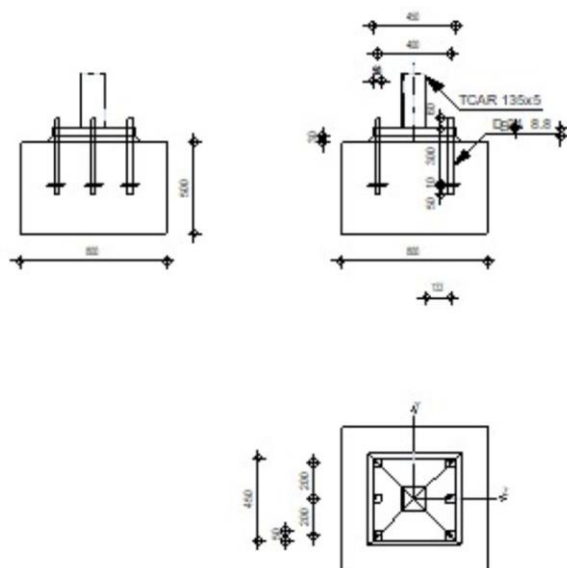
Autodesk Robot Structural Analysis Professional 2021

Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009 + CEB Design Guide:

Design of fastenings in concrete

OK

Ratio
0,95**GENERAL**

Connection no.: 56

Connection name: Fixed column base

Structure node: 7

Structure bars: 4

GEOMETRY**COLUMN**

Section: TCAR 135x5

Bar no.: 4

$L_c =$	2,40	[m]	Column length
$\alpha =$	0,0	[Deg]	Inclination angle
$h_c =$	135	[mm]	Height of column section
$b_{fc} =$	135	[mm]	Width of column section
$t_{wc} =$	5	[mm]	Thickness of the web of column section
$t_{fc} =$	5	[mm]	Thickness of the flange of column section
$r_c =$	11	[mm]	Radius of column section fillet
$A_c =$	25,10	[cm ²]	Cross-sectional area of a column
$I_{yc} =$	695,00	[cm ⁴]	Moment of inertia of the column section

Material: S275

 $f_{yc} = 275,00$ [MPa] Resistance $f_{uc} = 430,00$ [MPa] Yield strength of a material**COLUMN BASE** $l_{pd} = 450$ [mm] Length $b_{pd} = 450$ [mm] Width $t_{pd} = 40$ [mm] Thickness

Material: S275

 $f_{ypd} = 275,00$ [MPa] Resistance $f_{upd} = 430,00$ [MPa] Yield strength of a material**ANCHORAGE**

The shear plane passes through the UNTHREADED portion of the bolt.

Class = 8.8 Anchor class

 $f_{yb} = 640,00$ [MPa] Yield strength of the anchor material $f_{ub} = 800,00$ [MPa] Tensile strength of the anchor material $d = 24$ [mm] Bolt diameter $A_s = 3,53$ [cm²] Effective section area of a bolt $A_v = 4,52$ [cm²] Area of bolt section $n_H = 2$ Number of bolt columns $n_V = 3$ Number of bolt rowsHorizontal spacing $e_{Hi} = 400$ [mm]Vertical spacing $e_{Vi} = 200$ [mm]**Anchor dimensions** $L_1 = 60$ [mm] $L_2 = 300$ [mm] $L_3 = 50$ [mm]**Anchor plate** $l_p = 100$ [mm] Length $b_p = 100$ [mm] Width $t_p = 10$ [mm] Thickness

Material: S275

 $f_y = 275,00$ [MPa] Resistance**Washer**

$l_{wd} = 50$ [mm] Length
 $b_{wd} = 50$ [mm] Width
 $t_{wd} = 10$ [mm] Thickness

MATERIAL FACTORS

$\gamma_{M0} = 1,00$ Partial safety factor
 $\gamma_{M2} = 1,25$ Partial safety factor
 $\gamma_C = 1,50$ Partial safety factor

SPREAD FOOTING

$L = 800$ [mm] Spread footing length
 $B = 800$ [mm] Spread footing width
 $H = 500$ [mm] Spread footing height

Concrete

Class C25/30

$f_{ck} = 25,00$ [MPa] Characteristic resistance for compression

Grout layer

$t_g = 30$ [mm] Thickness of leveling layer (grout)

$f_{ck,g} = 12,00$ [MPa] Characteristic resistance for compression

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete

WELDS

$a_p = 5$ [mm] Footing plate of the column base

LOADS

Case: 18: 1 * X -1 * Y SQRT(11*1.00;12*-1.00)

$N_{j,Ed} = -2,67$ [kN] Axial force
 $V_{j,Ed,y} = 6,70$ [kN] Shear force
 $V_{j,Ed,z} = 6,19$ [kN] Shear force
 $M_{j,Ed,y} = 8,98$ [kN*m] Bending moment
 $M_{j,Ed,z} = 10,80$ [kN*m] Bending moment

RESULTS

COMPRESSION ZONE

COMPRESSION OF CONCRETE

$f_{cd} = 16,67$ [MPa] Design compressive resistance EN 1992-1-1:3.1.6.(1)

$f_j = 19,75$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

$$c = t_p \sqrt{(f_{yp} / (3 * f_j * \gamma_{M0}))}$$

$c = 86$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$A_{c0} = 545,02$ [cm²] Area of the joint between the base plate and the foundation EN 1992-1-1:6.7.(3)

$A_{c1} = 4256,09$ [cm²] Maximum design area of load distribution EN 1992-1-1:6.7.(3)

$$F_{rd,u} = A_{c0} * f_{cd} * \sqrt{(A_{c1} / A_{c0})} \leq 3 * A_{c0} * f_{cd}$$

$F_{rd,u} = 2538,40$ [kN] Bearing resistance of concrete EN 1992-1-1:6.7.(3)

$\beta_j = 0,67$ Reduction factor for compression [6.2.5.(7)]

$$f_{jd} = \beta_j * F_{rd,u} / (b_{eff} * l_{eff})$$

$f_{jd} = 31,05$ [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n} = 877,03$ [cm²] Bearing area for compression [6.2.8.2.(1)]

$A_{c,y} = 438,51$ [cm²] Bearing area for bending My [6.2.8.3.(1)]

$A_{c,z} = 438,51$ [cm²] Bearing area for bending Mz [6.2.8.3.(1)]

$$F_{c,Rd,i} = A_{c,i} * f_{jd}$$

$F_{c,Rd,n} = 2723,13$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]

$F_{c,Rd,y} = 1361,57$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

$F_{c,Rd,z} = 1361,57$ [kN] Bearing resistance of concrete for bending Mz [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL = 1,00$ Section class EN 1993-1-1:5.5.2

$W_{pl,y} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,y} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,y} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$$

$F_{c,fc,Rd,y} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

$W_{pl,z} = 102,96$ [cm³] Plastic section modulus EN1993-1-1:6.2.5.(2)

$M_{c,Rd,z} = 28,31$ [kN*m] Design resistance of the section for bending EN1993-1-1:6.2.5

$h_{f,z} = 130$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$$F_{c,fc,Rd,z} = M_{c,Rd,z} / h_{f,z}$$

$F_{c,fc,Rd,z} = 217,81$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$$N_{j,Rd} = F_{c,Rd,n}$$

$N_{j,Rd} = 2723,13$ [kN] Resistance of a spread footing for axial compression [6.2.8.2.(1)]

$$F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$$

$$F_{C,Rd,y} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

$$F_{C,Rd,z} = \min(F_{c,Rd,z}, F_{c,fc,Rd,z})$$

$$F_{C,Rd,z} = 217,81 \text{ [kN]} \quad \text{Resistance of spread footing in the compression zone} \quad [6.2.8.3]$$

TENSION ZONE

STEEL FAILURE

$$A_b = 3,53 \text{ [cm}^2\text{]} \quad \text{Effective anchor area} \quad [\text{Table 3.4}]$$

$$f_{ub} = 800,00 \text{ [MPa]} \quad \text{Tensile strength of the anchor material} \quad [\text{Table 3.4}]$$

$$\text{Beta} = 0,85 \quad \text{Reduction factor of anchor resistance} \quad [3.6.1.(3)]$$

$$F_{t,Rd,s1} = \text{beta} \cdot 0.9 \cdot f_{ub} \cdot A_b / \gamma_{M2}$$

$$F_{t,Rd,s1} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad [\text{Table 3.4}]$$

$$\gamma_{Ms} = 1,20 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.2]}$$

$$f_{yb} = 640,00 \text{ [MPa]} \quad \text{Yield strength of the anchor material} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s2} = f_{yb} \cdot A_b / \gamma_{Ms}$$

$$F_{t,Rd,s2} = 188,27 \text{ [kN]} \quad \text{Anchor resistance to steel failure} \quad \text{CEB [9.2.2]}$$

$$F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$$

$$F_{t,Rd,s} = 172,83 \text{ [kN]} \quad \text{Anchor resistance to steel failure}$$

PULL-OUT FAILURE

$$f_{ck} = 25,00 \text{ [MPa]} \quad \text{Characteristic compressive strength of concrete} \quad \text{EN 1992-1:[3.1.2]}$$

$$A_h = 95,48 \text{ [cm}^2\text{]} \quad \text{Bearing area of the head} \quad \text{CEB [15.1.2.3]}$$

$$p_k = 175,00 \text{ [MPa]} \quad \text{Characteristic strength of concrete (pull-out)} \quad \text{CEB [15.1.2.3]}$$

$$\gamma_{Mp} = 2,16 \quad \text{Partial safety factor} \quad \text{CEB [3.2.3.1]}$$

$$F_{t,Rd,p} = p_k \cdot A_h / \gamma_{Mp}$$

$$F_{t,Rd,p} = 828,79 \text{ [kN]} \quad \text{Design uplift capacity} \quad \text{CEB [9.2.3]}$$

CONCRETE CONE FAILURE

$$h_{ef} = 133 \text{ [mm]} \quad \text{Effective anchorage depth} \quad \text{CEB [9.2.4]}$$

$$N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$$

$$N_{Rk,c}^0 = 69,28 \text{ [kN]} \quad \text{Characteristic resistance of an anchor} \quad \text{CEB [9.2.4]}$$

$$s_{cr,N} = 400 \text{ [mm]} \quad \text{Critical width of the concrete cone} \quad \text{CEB [9.2.4]}$$

$$c_{cr,N} = 200 \text{ [mm]} \quad \text{Critical edge distance} \quad \text{CEB [9.2.4]}$$

$$A_{c,N0} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Maximum area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$A_{c,N} = 1600,00 \text{ [cm}^2\text{]} \quad \text{Actual area of concrete cone} \quad \text{CEB [9.2.4]}$$

$$\psi_{A,N} = A_{c,N} / A_{c,N0}$$

$\psi_{A,N} = 1,00$ Factor related to anchor spacing and edge distance CEB [9.2.4]
 $c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.4]
 $\psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$
 $\psi_{s,N} = 1,00$ Factor taking account the influence of edges of the concrete member on the distribution of stresses
 $\psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors
 $\psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$
 $\psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.5]
 $\psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]
 $\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.1]
 $F_{t,Rd,c} = N_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{ucr,N} / \gamma_{Mc}$
 $F_{t,Rd,c} = 32,08$ [kN] Design anchor resistance to concrete cone failure EN 1992-1-1:8.4.2.(2)

SPLITTING FAILURE

$h_{ef} = 270$ [mm] Effective anchorage depth CEB [9.2.5]
 $N_{Rk,c}^0 = 9.0 [N^{0.5}/mm^{0.5}] \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5}$
 $N_{Rk,c}^0 = 199,64$ [kN] Design uplift capacity CEB [9.2.5]
 $s_{cr,N} = 540$ [mm] Critical width of the concrete cone CEB [9.2.5]
 $c_{cr,N} = 270$ [mm] Critical edge distance CEB [9.2.5]
 $A_{c,N0} = 2916,00$ [cm²] Maximum area of concrete cone CEB [9.2.5]
 $A_{c,N} = 1880,00$ [cm²] Actual area of concrete cone CEB [9.2.5]
 $\psi_{A,N} = A_{c,N}/A_{c,N0}$
 $\psi_{A,N} = 0,64$ Factor related to anchor spacing and edge distance CEB [9.2.5]
 $c = 200$ [mm] Minimum edge distance from an anchor CEB [9.2.5]
 $\psi_{s,N} = 0.7 + 0.3 \cdot c/c_{cr,N} \leq 1.0$
 $\psi_{s,N} = 0,92$ Factor taking account the influence of edges of the concrete member on the distribution of stresses
 $\psi_{ec,N} = 1,00$ Factor related to distribution of tensile forces acting on anchors
 $\psi_{re,N} = 0.5 + h_{ef}/200 \leq 1.0$
 $\psi_{re,N} = 1,00$ Shell spalling factor CEB [9.2.5]
 $\psi_{ucr,N} = 1,00$ Factor taking into account whether the anchorage is in cracked or non-cracked concrete CEB [9.2.5]
 $\psi_{h,N} = (h/(2 \cdot h_{ef}))^{2/3} \leq 1.2$
 $\psi_{h,N} = 0,95$ Coeff. related to the foundation height CEB [9.2.5]
 $\gamma_{M,sp} = 2,16$ Partial safety factor CEB [3.2.3.1]
 $F_{t,Rd,sp} = N_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{ucr,N} \cdot \psi_{h,N} / \gamma_{M,sp}$
 $F_{t,Rd,sp} = 52,21$ [kN] Design anchor resistance to splitting of concrete CEB [9.2.5]

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$
 $F_{t,Rd} = 32,08$ [kN] Tensile resistance of an anchor

BENDING OF THE BASE PLATE**Bending moment $M_{j,Ed,y}$**

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 780,49$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 341,84$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 96,23$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,y} = 96,23$ [kN] Tension resistance of a plate [6.2.4]

Bending moment $M_{j,Ed,z}$

$l_{eff,1} = 225$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]

$l_{eff,2} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$M_{pl,1,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]

$M_{pl,2,Rd} = 24,75$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]

$F_{T,1,Rd} = 528,33$ [kN] Resistance of a plate for mode 1 [6.2.4]

$F_{T,2,Rd} = 232,42$ [kN] Resistance of a plate for mode 2 [6.2.4]

$F_{T,3,Rd} = 64,15$ [kN] Resistance of a plate for mode 3 [6.2.4]

$F_{t,pl,Rd,z} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$F_{t,pl,Rd,z} = 64,15$ [kN] Tension resistance of a plate [6.2.4]

TENSILE RESISTANCE OF A COLUMN WEB**Bending moment $M_{j,Ed,y}$**

$t_{wc} = 5$ [mm] Effective thickness of the column web [6.2.6.3.(8)]

$b_{eff,t,wc} = 225$ [mm] Effective width of the web for tension [6.2.6.3.(2)]

$A_{vc} = 12,55$ [cm²] Shear area EN1993-1-1:[6.2.6.(3)]

$\omega = 0,44$ Reduction factor for interaction with shear [6.2.6.3.(4)]

$F_{t,wc,Rd,y} = \omega b_{eff,t,wc} t_{wc} f_{yc} / \gamma_{M0}$

$F_{t,wc,Rd,y} = 271,90$ [kN] Column web resistance [6.2.6.3.(1)]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$F_{T,Rd,y} = \min(F_{t,pl,Rd,y}, F_{t,wc,Rd,y})$

$F_{T,Rd,y} = 96,23$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

$F_{T,Rd,z} = F_{t,pl,Rd,z}$

$F_{T,Rd,z} = 64,15$ [kN] Resistance of a column base in the tension zone [6.2.8.3]

CONNECTION CAPACITY CHECK

$$N_{j,Ed} / N_{j,Rd} \leq 1,0 \text{ (6.24)} \quad 0,00 < 1,00 \quad \text{verified} \quad (0,00)$$

$$e_y = 3367 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 84 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 27,99 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \text{ (6.23)} \quad 0,32 < 1,00 \quad \text{verified} \quad (0,32)$$

$$e_z = 4049 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,z} = 65 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,z} \quad [6.2.8.1.(2)]$$

$$z_{t,z} = 200 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,z} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,z} = 17,28 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \text{ (6.23)} \quad 0,63 < 1,00 \quad \text{verified} \quad (0,63)$$

$$M_{j,Ed,y} / M_{j,Rd,y} + M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0 \quad 0,95 < 1,00 \quad \text{verified} \quad (0,95)$$

SHEAR**BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE****Shear force $V_{j,Ed,y}$**

$$\alpha_{d,y} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,y} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,y} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,y} = k_{1,y} * \alpha_{b,y} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,y} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

Shear force $V_{j,Ed,z}$

$$\alpha_{d,z} = 0,32 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0,32 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 0,99 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

$$F_{1,vb,Rd,z} = k_{1,z} * \alpha_{b,z} * f_{up} * d * t_p / \gamma_{M2}$$

$$F_{1,vb,Rd,z} = 105,03 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

$\alpha_b = 0,25$ Coeff. for resistance calculation $F_{2,vb,Rd}$ [6.2.2.(7)]

$A_{vb} = 4,52 \text{ [cm}^2\text{]}$ Area of bolt section [6.2.2.(7)]

$f_{ub} = 800,00 \text{ [MPa]}$ Tensile strength of the anchor material [6.2.2.(7)]

$\gamma_{M2} = 1,25$ Partial safety factor [6.2.2.(7)]

$$F_{2,vb,Rd} = \alpha_b * f_{ub} * A_{vb} / \gamma_{M2}$$

$F_{2,vb,Rd} = 71,80 \text{ [kN]}$ Shear resistance of a bolt - without lever arm [6.2.2.(7)]

$\alpha_M = 2,00$ Factor related to the fastening of an anchor in the foundation CEB [9.3.2.2]

$M_{Rk,s} = 1,10 \text{ [kN*m]}$ Characteristic bending resistance of an anchor CEB [9.3.2.2]

$l_{sm} = 62 \text{ [mm]}$ Lever arm length CEB [9.3.2.2]

$\gamma_{Ms} = 1,20$ Partial safety factor CEB [3.2.3.2]

$$F_{v,Rd,sm} = \alpha_M * M_{Rk,s} / (l_{sm} * \gamma_{Ms})$$

$F_{v,Rd,sm} = 29,46 \text{ [kN]}$ Shear resistance of a bolt - with lever arm CEB [9.3.1]

CONCRETE PRY-OUT FAILURE

$N_{Rk,c} = 69,28 \text{ [kN]}$ Design uplift capacity CEB [9.2.4]

$k_3 = 2,00$ Factor related to the anchor length CEB [9.3.3]

$\gamma_{Mc} = 2,16$ Partial safety factor CEB [3.2.3.1]

$$F_{v,Rd,cp} = k_3 * N_{Rk,c} / \gamma_{Mc}$$

$F_{v,Rd,cp} = 64,15 \text{ [kN]}$ Concrete resistance for pry-out failure CEB [9.3.1]

CONCRETE EDGE FAILURE

Shear force $V_{j,Ed,y}$

$V_{Rk,c,y}^0 = 70,02 \text{ [kN]}$ Characteristic resistance of an anchor

$\psi_{A,V,y} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,y} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,y} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,y} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,y} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,y} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$$F_{v,Rd,c,y} = V_{Rk,c,y}^0 * \psi_{A,V,y} * \psi_{h,V,y} * \psi_{s,V,y} * \psi_{ec,V,y} * \psi_{\alpha,V,y} * \psi_{ucr,V,y} / \gamma_{Mc}$$

$F_{v,Rd,c,y} = 19,45 \text{ [kN]}$ Concrete resistance for edge failure CEB [9.3.1]

Shear force $V_{j,Ed,z}$

$V_{Rk,c,z}^0 = 70,02$ [kN] Characteristic resistance of an anchor

$\psi_{A,V,z} = 0,67$ Factor related to anchor spacing and edge distance

$\psi_{h,V,z} = 1,00$ Factor related to the foundation thickness

$\psi_{s,V,z} = 0,90$ Factor related to the influence of edges parallel to the shear load direction

$\psi_{ec,V,z} = 1,00$ Factor taking account a group effect when different shear loads are acting on the individual a

$\psi_{\alpha,V,z} = 1,00$ Factor related to the angle at which the shear load is applied

$\psi_{ucr,V,z} = 1,00$ Factor related to the type of edge reinforcement used

$\gamma_{Mc} = 2,16$ Partial safety factor

$F_{v,Rd,c,z} = V_{Rk,c,z}^0 \psi_{A,V,z} \psi_{h,V,z} \psi_{s,V,z} \psi_{ec,V,z} \psi_{\alpha,V,z} \psi_{ucr,V,z} / \gamma_{Mc}$

$F_{v,Rd,c,z} = 19,45$ [kN] Concrete resistance for edge failure CEB [9.3.1]

SPLITTING RESISTANCE

$C_{f,d} = 0,30$ Coeff. of friction between the base plate and concrete [6.2.2.(6)]

$N_{c,Ed} = 2,67$ [kN] Compressive force [6.2.2.(6)]

$F_{f,Rd} = C_{f,d} N_{c,Ed}$

$F_{f,Rd} = 0,80$ [kN] Slip resistance [6.2.2.(6)]

SHEAR CHECK

$V_{j,Rd,y} = n_b \min(F_{1,vb,Rd,y}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{f,Rd}$

$V_{j,Rd,y} = 117,51$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,y} / V_{j,Rd,y} \leq 1,0$ $0,06 < 1,00$ verified (0,06)

$V_{j,Rd,z} = n_b \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$

$V_{j,Rd,z} = 117,51$ [kN] Connection resistance for shear CEB [9.3.1]

$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,05 < 1,00$ verified (0,05)

$V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$ $0,11 < 1,00$ verified (0,11)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp} = 117,55$ [MPa] Normal stress in a weld [4.5.3.(7)]

$\tau_{\perp} = 117,55$ [MPa] Perpendicular tangent stress [4.5.3.(7)]

$\tau_{yII} = 4,96$ [MPa] Tangent stress parallel to $V_{j,Ed,y}$ [4.5.3.(7)]

$\tau_{zII} = 4,59$ [MPa] Tangent stress parallel to $V_{j,Ed,z}$ [4.5.3.(7)]

$\beta_W = 0,85$ Resistance-dependent coefficient [4.5.3.(7)]

$\sigma_{\perp} / (0,9 f_u / \gamma_{M2}) \leq 1,0$ (4.1) $0,38 < 1,00$ verified (0,38)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{yII}^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \gamma_{M2}))} \leq 1,0$ (4.1) $0,58 < 1,00$ verified (0,58)

$\sqrt{(\sigma_{\perp}^2 + 3,0 (\tau_{zII}^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \gamma_{M2}))} \leq 1,0$ (4.1) $0,55 < 1,00$ verified (0,55)

CONNECTION STIFFNESS**Bending moment $M_{j,Ed,y}$**

$b_{eff} = 177$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 307$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff}) / (1.275 \cdot E)}$$

$k_{13,y} = 27$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 127$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,y} = 6$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,y} = 1.6 \cdot A_b / L_b$$

$k_{16,y} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,y} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,y} = 24414,02$ [kN*m] Initial rotational stiffness [Table 6.12]

$S_{j,rig,y} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,y} \geq S_{j,rig,y}$ RIGID [5.2.2.5.(2)]

Bending moment $M_{j,Ed,z}$

$$k_{13,z} = E_c \cdot \sqrt{(A_{c,z}) / (1.275 \cdot E)}$$

$k_{13,z} = 24$ [mm] Stiffness coeff. of compressed concrete [Table 6.11]

$l_{eff} = 225$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]

$m = 187$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]

$$k_{15,z} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$$

$k_{15,z} = 2$ [mm] Stiffness coeff. of the base plate subjected to tension [Table 6.11]

$L_b = 284$ [mm] Effective anchorage depth [Table 6.11]

$$k_{16,z} = 1.6 \cdot A_b / L_b$$

$k_{16,z} = 2$ [mm] Stiffness coeff. of an anchor subjected to tension [Table 6.11]

$\lambda_{0,z} = 0,53$ Column slenderness [5.2.2.5.(2)]

$S_{j,ini,z} = 13821,83$ [kN*m] Initial rotational stiffness [6.3.1.(4)]

$S_{j,rig,z} = 18243,75$ [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,ini,z} < S_{j,rig,z}$ SEMI-RIGID [5.2.2.5.(2)]

WEAKEST COMPONENT:

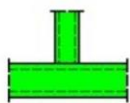
BASE PLATE - BENDING

REMARKS

Horizontal distance bolt-plate edge is too small. 25 [mm] < 29 [mm]

Connection conforms to the code

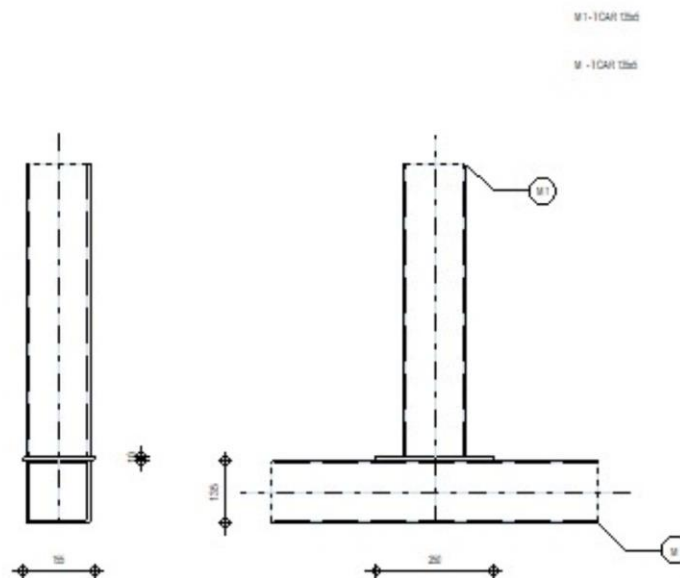
Ratio 0,95



Autodesk Robot Structural Analysis Professional 2021

Design of truss node connection

EN 1993-1-8:2005/AC:2009

OKRatio
0,78**GENERAL**

Connection no.: 3

Connection name: Tube

Structure node: 132

Structure bars: 13, 12

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		13			12	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3880			3940	mm

HORIZONTAL BRACKETb_{ph} = 155 [mm] Widthl_{ph} = 250 [mm] Lengtht_{ph} = 10 [mm] Thickness

Material: Def

f_{yph} = 235,00 [MPa] Resistance**WELDS**a_d = 5 [mm] Thickness of welds of diagonals and postsa_{st} = 5 [mm] Thickness of stiffener welds**LOADS**

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORDN_{01,Ed} = -5,18 [kN] Axial forceM_{01,Ed} = -29,95 [kN*m] Bending momentN_{02,Ed} = -5,18 [kN] Axial forceM_{02,Ed} = -29,95 [kN*m] Bending moment**POST**

$N_3 = -1,47$ [kN] Axial force

$M_3 = 6,79$ [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS

CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

$\beta = 1,00$ Coefficient taking account of geometry of connection bars

$\beta = b_3/b_0$ [1.5 (6)]

$\gamma = 6,75$ Coefficient taking account of geometry of the chord

$\gamma = b_0/(2*t_{ph})$ [1.5 (6)]

TUBE BRACE FAILURE

POST

$W_{pl,3} = 102,96$ [cm³] Plastic section modulus

$b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$

$N_{3,Rd} = 715,00$ [kN] Compression capacity

$N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-1,47| < 715,00$ **verified** (0,00)

$M_{3,Rd} = 13,12$ [kN*m] Bending resistance

$M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$

$|M_3| \leq M_{3,Rd}$ $|6,79| < 13,12$ **verified** (0,52)

$N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$

$0,52 < 1,00$

verified

(0,52)

CHORD SIDE WALL BUCKLING

Post

$f_b = 149,06$ [MPa] Buckling strength of the chord side wall

$f_b = \chi * f_{y0}$

$N_{3,Rd} = 208,42$ [kN] Compression capacity

$N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-1,47| < 208,42$ **verified** (0,01)

CHORD SHEAR**POST**

$$A_v = 13,50 \text{ [cm}^2\text{]} \quad \text{Shear area of the chord} \quad A_v = 2 \cdot h_0 \cdot t_0$$

$$N_{3,Rd} = 214,34 \text{ [kN]} \quad \text{Compression capacity} \quad N_{3,Rd} = f_{y0} \cdot A_v [\sqrt{3} \sin(\theta_3)] / \gamma_{M5}$$

$$|N_3| \leq N_{3,Rd} \quad |-1,47| < 214,34 \quad \text{verified} \quad (0,01)$$

CHORD RESISTANCE

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Compression capacity} \quad N_{0,Rd} = (A_0 \cdot f_{y0}) / \gamma_{M5}$$

$$|N_{01}| \leq N_{0,Rd} \quad |-5,18| < 690,25 \quad \text{verified} \quad (0,01)$$

CHORD SIDE WALL CRUSHING**POST**

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 \cdot f_{y0} \cdot t_0 \cdot (h_3 + 5 \cdot t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |6,79| < 17,60 \quad \text{verified} \quad (0,39)$$

VERIFICATION OF WELDS**POST**

$$\beta_w = 0,85 \quad \text{Correlation coefficient} \quad [\text{Table 4.1}]$$

$$\gamma_{M2} = 1,25 \quad \text{Partial safety factor} \quad [\text{Table 2.1}]$$

Longitudinal weld

$$\sigma_{\perp} = -158,73 \text{ [MPa]} \quad \text{Normal stress in a weld}$$

$$\tau_{\perp} = -158,73 \text{ [MPa]} \quad \text{Perpendicular tangent stress}$$

$$\tau_{\parallel} = -0,00 \text{ [MPa]} \quad \text{Tangent stress}$$

$$|\sigma_{\perp}| \leq 0.9 \cdot f_u / \gamma_{M2} \quad |-158,73| < 309,60 \quad \text{verified} \quad (0,51)$$

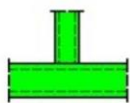
$$\sqrt{[\sigma_{\perp}]^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)} \leq f_u / (\beta_w \cdot \gamma_{M2}) \quad 317,45 < 404,71 \quad \text{verified} \quad (0,78)$$

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

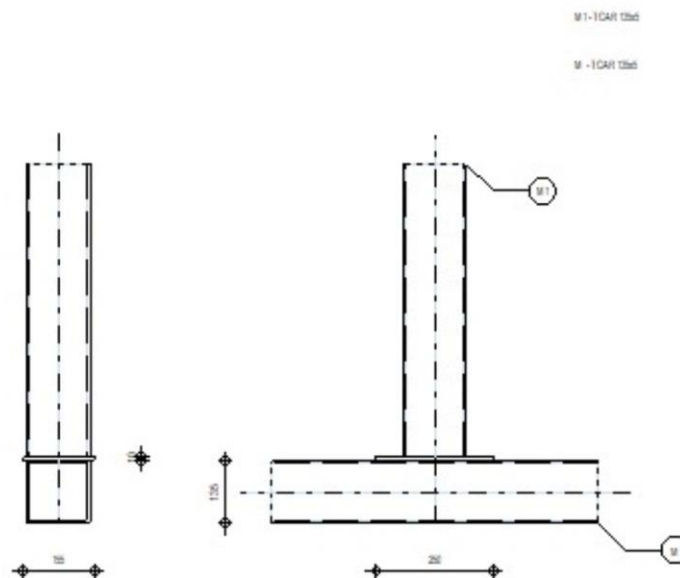
Ratio 0,78



Autodesk Robot Structural Analysis Professional 2021

Design of truss node connection

EN 1993-1-8:2005/AC:2009

Ratio
0,19**GENERAL**

Connection no.: 4
 Connection name: Column-Beam
 Structure node: 131
 Structure bars: 10, 6

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		10			6	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3940			2400	mm

HORIZONTAL BRACKETb_{ph} = 155 [mm] Widthl_{ph} = 250 [mm] Lengtht_{ph} = 10 [mm] Thickness

Material: Def

f_{yph} = 235,00 [MPa] Resistance**WELDS**a_d = 8 [mm] Thickness of welds of diagonals and postsa_{st} = 5 [mm] Thickness of stiffener welds**LOADS**

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORDN_{01,Ed} = -5,39 [kN] Axial forceM_{01,Ed} = 7,99 [kN*m] Bending momentN_{02,Ed} = -4,02 [kN] Axial forceM_{02,Ed} = 8,02 [kN*m] Bending moment**POST**

$N_3 = -39,71$ [kN] Axial force

$M_3 = 0,00$ [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS

CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

$\beta = 1,00$ Coefficient taking account of geometry of connection bars

$\beta = b_3/b_0$ [1.5 (6)]

$\gamma = 6,75$ Coefficient taking account of geometry of the chord

$\gamma = b_0/(2*t_{ph})$ [1.5 (6)]

TUBE BRACE FAILURE

POST

$W_{pl,3} = 102,96$ [cm³] Plastic section modulus

$b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$

$N_{3,Rd} = 715,00$ [kN] Compression capacity

$N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-39,71| < 715,00$ **verified** (0,06)

$M_{3,Rd} = 13,12$ [kN*m] Bending resistance

$M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$

$|M_3| \leq M_{3,Rd}$ $|0,00| < 13,12$ **verified** (0,00)

$N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$

$0,06 < 1,00$

verified

(0,06)

CHORD SIDE WALL BUCKLING

Post

$f_b = 149,06$ [MPa] Buckling strength of the chord side wall

$f_b = \chi * f_{y0}$

$N_{3,Rd} = 238,50$ [kN] Compression capacity

$N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-39,71| < 238,50$ **verified** (0,17)

CHORD SHEAR

Post

$A_v =$	13,50	[cm ²]	Shear area of the chord	$A_v = 2 \cdot h_0 \cdot t_0$
$N_{3,Rd} =$	214,34	[kN]	Compression capacity	$N_{3,Rd} = f_{y0} \cdot A_v \cdot [\sqrt{3} \cdot \sin(\theta_3)] / \gamma_{M5}$
$ N_3 \leq N_{3,Rd}$	-39,71 < 214,34		verified	(0,19)

CHORD RESISTANCE

$N_{0,Rd} =$	690,25	[kN]	Compression capacity	$N_{0,Rd} = (A_0 \cdot f_{y0}) / \gamma_{M5}$
$ N_{01} \leq N_{0,Rd}$	-5,39 < 690,25		verified	(0,01)

CHORD SIDE WALL CRUSHING

Post

$M_{3,Rd} =$	17,60	[kN*m]	Bending resistance	$M_{3,Rd} = 0.5 \cdot f_{y0} \cdot t_0 \cdot (h_3 + 5 \cdot t_0)^2 / \gamma_{M5}$
$ M_3 \leq M_{3,Rd}$	0,00 < 17,60		verified	(0,00)

VERIFICATION OF WELDS

Post

$\beta_w =$	0,85	Correlation coefficient	[Table 4.1]
$\gamma_{M2} =$	1,25	Partial safety factor	[Table 2.1]

Longitudinal weld

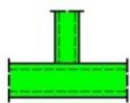
$\sigma_{\perp} =$	-13,00	[MPa]	Normal stress in a weld	
$\tau_{\perp} =$	-13,00	[MPa]	Perpendicular tangent stress	
$\tau_{ } =$	-0,00	[MPa]	Tangent stress	
$ \sigma_{\perp} \leq 0.9 \cdot f_u / \gamma_{M2}$	-13,00 < 309,60		verified	(0,04)
$\sqrt{[\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{ }^2)]} \leq f_u / (\beta_w \cdot \gamma_{M2})$	26,00 < 404,71		verified	(0,06)

REMARKS

Ratio of the post width to the chord width is too large 1,00 > 0,85

Connection conforms to the code

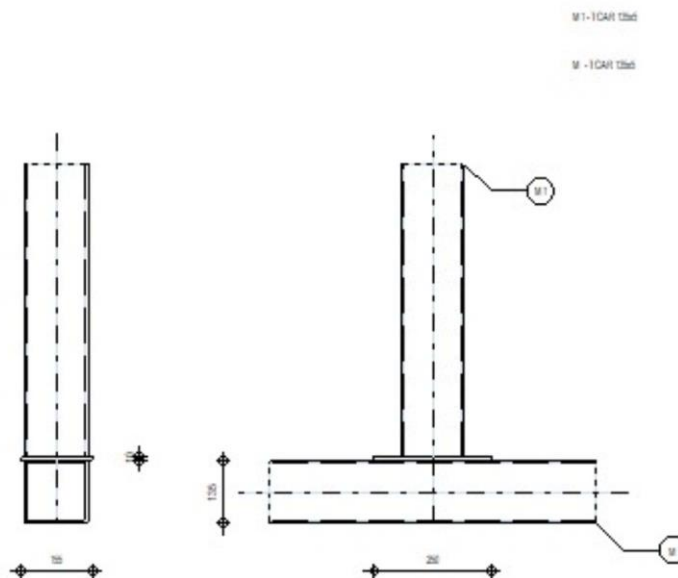
Ratio 0,19



Autodesk Robot Structural Analysis Professional 2021

Design of truss node connection

EN 1993-1-8:2005/AC:2009

Ratio
0,19**GENERAL**

Connection no.: 5
 Connection name: Column-Beam
 Structure node: 12
 Structure bars: 7, 5

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		7			5	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3940			2400	mm

HORIZONTAL BRACKETb_{ph} = 155 [mm] Widthl_{ph} = 250 [mm] Lengtht_{ph} = 10 [mm] Thickness

Material: Def

f_{yph} = 235,00 [MPa] Resistance**WELDS**a_d = 5 [mm] Thickness of welds of diagonals and postsa_{st} = 5 [mm] Thickness of stiffener welds**LOADS**

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORDN_{01,Ed} = -4,02 [kN] Axial forceM_{01,Ed} = 8,02 [kN*m] Bending momentN_{02,Ed} = -5,39 [kN] Axial forceM_{02,Ed} = 7,99 [kN*m] Bending moment**POST**

$N_3 = -39,71$ [kN] Axial force

$M_3 = 0,00$ [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS

CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

$\beta = 1,00$ Coefficient taking account of geometry of connection bars

$\beta = b_3/b_0$ [1.5 (6)]

$\gamma = 6,75$ Coefficient taking account of geometry of the chord

$\gamma = b_0/(2*t_{ph})$ [1.5 (6)]

TUBE BRACE FAILURE

POST

$W_{pl,3} = 102,96$ [cm³] Plastic section modulus

$b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$

$N_{3,Rd} = 715,00$ [kN] Compression capacity

$N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-39,71| < 715,00$ **verified** (0,06)

$M_{3,Rd} = 13,12$ [kN*m] Bending resistance

$M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$

$|M_3| \leq M_{3,Rd}$ $|0,00| < 13,12$ **verified** (0,00)

$N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$ $0,06 < 1,00$ **verified** (0,06)

CHORD SIDE WALL BUCKLING

Post

$f_b = 149,06$ [MPa] Buckling strength of the chord side wall

$f_b = \chi * f_{y0}$

$N_{3,Rd} = 238,50$ [kN] Compression capacity

$N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-39,71| < 238,50$ **verified** (0,17)

CHORD SHEAR

Post

$A_v =$	13,50	[cm ²]	Shear area of the chord	$A_v = 2 \cdot h_0 \cdot t_0$
$N_{3,Rd} =$	214,34	[kN]	Compression capacity	$N_{3,Rd} = f_{y0} \cdot A_v \cdot [\sqrt{3} \cdot \sin(\theta_3)] / \gamma_{M5}$
$ N_3 \leq N_{3,Rd}$	-39,71 < 214,34		verified	(0,19)

CHORD RESISTANCE

$N_{0,Rd} =$	690,25	[kN]	Compression capacity	$N_{0,Rd} = (A_0 \cdot f_{y0}) / \gamma_{M5}$
$ N_{02} \leq N_{0,Rd}$	-5,39 < 690,25		verified	(0,01)

CHORD SIDE WALL CRUSHING

Post

$M_{3,Rd} =$	17,60	[kN*m]	Bending resistance	$M_{3,Rd} = 0.5 \cdot f_{y0} \cdot t_0 \cdot (h_3 + 5 \cdot t_0)^2 / \gamma_{M5}$
$ M_3 \leq M_{3,Rd}$	0,00 < 17,60		verified	(0,00)

VERIFICATION OF WELDS

Post

$\beta_w =$	0,85	Correlation coefficient	[Table 4.1]
$\gamma_{M2} =$	1,25	Partial safety factor	[Table 2.1]

Longitudinal weld

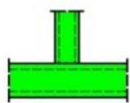
$\sigma_{\perp} =$	-20,80	[MPa]	Normal stress in a weld	
$\tau_{\perp} =$	-20,80	[MPa]	Perpendicular tangent stress	
$\tau_{ } =$	-0,00	[MPa]	Tangent stress	
$ \sigma_{\perp} \leq 0.9 \cdot f_u / \gamma_{M2}$	-20,80 < 309,60		verified	(0,07)
$\sqrt{[\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{ }^2)]} \leq f_u / (\beta_w \cdot \gamma_{M2})$	41,60 < 404,71		verified	(0,10)

REMARKS

Ratio of the post width to the chord width is too large 1,00 > 0,85

Connection conforms to the code

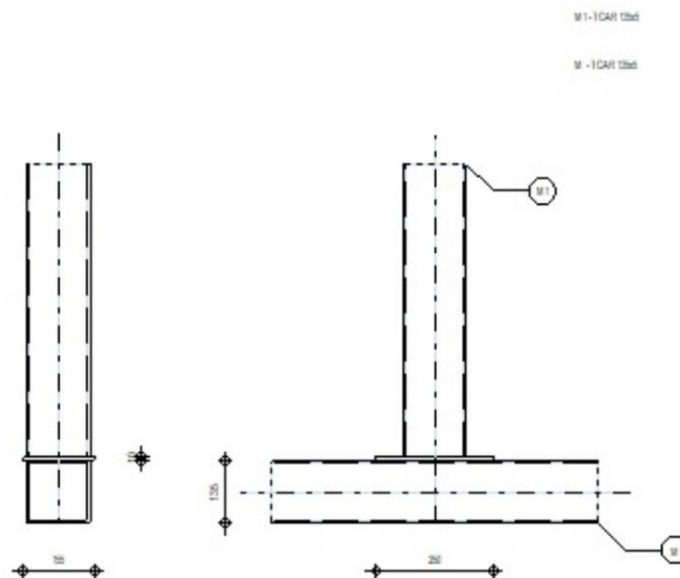
Ratio 0,19



Autodesk Robot Structural Analysis Professional 2021

Design of truss node connection

EN 1993-1-8:2005/AC:2009

Ratio
0,80**GENERAL**

Connection no.: 6

Connection name: Tube

Structure node: 9

Structure bars: 14, 11

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		14			11	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3880			3940	mm

HORIZONTAL BRACKETb_{ph} = 155 [mm] Widthl_{ph} = 250 [mm] Lengtht_{ph} = 10 [mm] Thickness

Material: Def

f_{yph} = 235,00 [MPa] Resistance**WELDS**a_d = 5 [mm] Thickness of welds of diagonals and postsa_{st} = 5 [mm] Thickness of stiffener welds**LOADS**

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORDN_{01,Ed} = -6,63 [kN] Axial forceM_{01,Ed} = -11,59 [kN*m] Bending momentN_{02,Ed} = -6,63 [kN] Axial forceM_{02,Ed} = -11,59 [kN*m] Bending moment**POST**

$N_3 = -2,28$ [kN] Axial force

$M_3 = 6,88$ [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS

CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

$\beta = 1,00$ Coefficient taking account of geometry of connection bars

$\beta = b_3/b_0$ [1.5 (6)]

$\gamma = 6,75$ Coefficient taking account of geometry of the chord

$\gamma = b_0/(2*t_{ph})$ [1.5 (6)]

TUBE BRACE FAILURE

POST

$W_{pl,3} = 102,96$ [cm³] Plastic section modulus

$b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$

$N_{3,Rd} = 715,00$ [kN] Compression capacity

$N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-2,28| < 715,00$ **verified** (0,00)

$M_{3,Rd} = 13,12$ [kN*m] Bending resistance

$M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$

$|M_3| \leq M_{3,Rd}$ $|6,88| < 13,12$ **verified** (0,52)

$N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$

$0,53 < 1,00$

verified

(0,53)

CHORD SIDE WALL BUCKLING

Post

$f_b = 149,06$ [MPa] Buckling strength of the chord side wall

$f_b = \chi * f_{y0}$

$N_{3,Rd} = 238,50$ [kN] Compression capacity

$N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-2,28| < 238,50$ **verified** (0,01)

CHORD SHEAR

Post

$A_v =$	13,50	[cm ²]	Shear area of the chord	$A_v = 2 \cdot h_0 \cdot t_0$
$N_{3,Rd} =$	214,34	[kN]	Compression capacity	$N_{3,Rd} = f_{y0} \cdot A_v \cdot [\sqrt{3} \cdot \sin(\theta_3)] / \gamma_{M5}$
$ N_3 \leq N_{3,Rd}$	-2,28 < 214,34		verified	(0,01)

CHORD RESISTANCE

$N_{0,Rd} =$	690,25	[kN]	Compression capacity	$N_{0,Rd} = (A_0 \cdot f_{y0}) / \gamma_{M5}$
$ N_{02} \leq N_{0,Rd}$	-6,63 < 690,25		verified	(0,01)

CHORD SIDE WALL CRUSHING

Post

$M_{3,Rd} =$	17,60	[kN*m]	Bending resistance	$M_{3,Rd} = 0.5 \cdot f_{y0} \cdot t_0 \cdot (h_3 + 5 \cdot t_0)^2 / \gamma_{M5}$
$ M_3 \leq M_{3,Rd}$	6,88 < 17,60		verified	(0,39)

VERIFICATION OF WELDS

Post

$\beta_w =$	0,85	Correlation coefficient	[Table 4.1]
$\gamma_{M2} =$	1,25	Partial safety factor	[Table 2.1]

Longitudinal weld

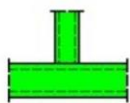
$\sigma_{\perp} =$	-161,27	[MPa]	Normal stress in a weld	
$\tau_{\perp} =$	-161,27	[MPa]	Perpendicular tangent stress	
$\tau_{\parallel} =$	-0,00	[MPa]	Tangent stress	
$ \sigma_{\perp} \leq 0.9 \cdot f_u / \gamma_{M2}$	-161,27 < 309,60		verified	(0,52)
$\sqrt{[\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w \cdot \gamma_{M2})$	322,55 < 404,71		verified	(0,80)

REMARKS

Ratio of the post width to the chord width is too large 1,00 > 0,85

Connection conforms to the code

Ratio 0,80

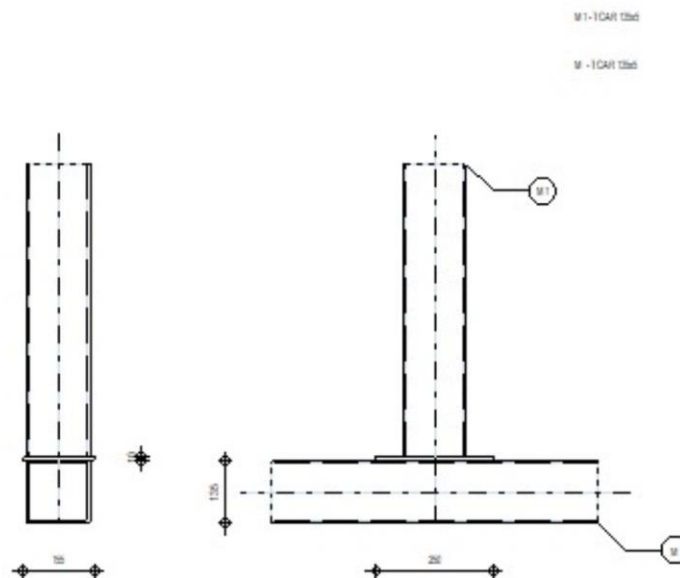


Autodesk Robot Structural Analysis Professional 2021

Design of truss node connection

EN 1993-1-8:2005/AC:2009

OK

Ratio
0,80**GENERAL**

Connection no.: 7
 Connection name: Tube
 Structure node: 10
 Structure bars: 15, 12

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		15			12	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3880			3940	mm

HORIZONTAL BRACKETb_{ph} = 155 [mm] Widthl_{ph} = 250 [mm] Lengtht_{ph} = 10 [mm] Thickness

Material: Def

f_{yph} = 235,00 [MPa] Resistance**WELDS**a_d = 5 [mm] Thickness of welds of diagonals and postsa_{st} = 5 [mm] Thickness of stiffener welds**LOADS**

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORDN_{01,Ed} = -8,49 [kN] Axial forceM_{01,Ed} = -10,94 [kN*m] Bending momentN_{02,Ed} = -8,49 [kN] Axial forceM_{02,Ed} = -10,94 [kN*m] Bending moment**POST**

$N_3 = 2,50$ [kN] Axial force
 $M_3 = 6,87$ [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS

CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005

$\gamma_{M5} = 1,00$ Partial safety factor [Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

$\beta = 0,87$ Coefficient taking account of geometry of connection bars $\beta = b_3/b_{ph}$ [1.5 (6)]
 $\gamma = 6,75$ Coefficient taking account of geometry of the chord $\gamma = b_0/(2*t_{ph})$ [1.5 (6)]
 $n = 0,40$ Coefficient taking account of stresses in the chord $n_0 = \sigma_{0,Ed}/f_{y0}$
 $k_n = 1,00$ Coefficient taking account of stresses in the chord $k_n = 1.0$

CHORD PUNCHING

Post

$b_{e,p} = 100$ [mm] Effective width for punching shear $b_{e,p} = (10*b_3)/(b_0/t_0)$

$N_{3,Rd} = 746,23$ [kN] Tension capacity $N_{3,Rd} = f_{y0} * t_{ph} / (\sqrt{3} * \sin(\theta_3)) * [2 * h_3 / \sin(\theta_3) + 2 * b_{e,p}] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|2,50| < 746,23$ **verified** (0,00)

TUBE BRACE FAILURE

Post

$W_{pl,3} = 102,96$ [cm³] Plastic section modulus

$b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$

$N_{3,Rd} = 715,00$ [kN] Tension capacity $N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|2,50| < 715,00$ **verified** (0,00)

$M_{3,Rd} = 13,12$ [kN*m] Bending resistance $M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$
 $|M_3| \leq M_{3,Rd}$ $|6,87| < 13,12$ **verified** (0,52)

$$N_{3,Rd}/N_{3,Rd} + M_{3,Rd}/M_{3,Rd} \leq 1 \quad 0,53 < 1,00 \quad \text{verified} \quad (0,53)$$

CHORD SIDE WALL CRUSHING

POST

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 \cdot f_{y0} \cdot t_0 \cdot (h_3 + 5 \cdot t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |6,87| < 17,60 \quad \text{verified} \quad (0,39)$$

CHORD RESISTANCE

$$V_{pl,Rd} = 214,34 \text{ [kN]} \quad \text{Plastic resistance for shear} \quad V_{pl,Rd} = (A_v \cdot f_{y0}) / (\sqrt{3} \cdot \gamma_{M0})$$

$$|V_{Ed}| \leq V_{pl,Rd} \quad |0,00| < 214,34 \quad \text{verified} \quad (0,00)$$

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Chord resistance} \quad N_{0,Rd} = (A_0 \cdot f_{y0}) / \gamma_{M5}$$

$$|N_{02}| \leq N_{0,Rd} \quad |-8,49| < 690,25 \quad \text{verified} \quad (0,01)$$

INTERPOLATION

POST

Tube chord face failure

$$N_{3,Rd(\beta=0.85)} = 509,04 \text{ [kN]} \quad \text{Tension capacity} \quad N_{3,Rd(\beta=0.85)} = [(k_n \cdot f_{yp} \cdot t_{ph}^2) / (1-\beta) \cdot \sin(\theta_3)] \cdot [2 \cdot \beta / \sin(\theta_3) + 4 \cdot \sqrt{(1-\beta)}] / \gamma_M$$

Chord side wall buckling

$$f_b = 149,06 \text{ [MPa]} \quad \text{Buckling strength of the chord side wall} \quad f_b = \chi \cdot f_{y0}$$

$$N_{3,Rd(\beta=1.0)} = 238,50 \text{ [kN]} \quad \text{Tension capacity} \quad N_{3,Rd(\beta=1.0)} = [(k_n \cdot f_b \cdot t_0) / \sin(\theta_3)] \cdot [(2 \cdot h_3) / \sin(\theta_3) + 10 \cdot t_0] / \gamma_{M5}$$

Chord shear

$$A_v = 13,50 \text{ [cm}^2\text{]} \quad \text{Shear area of the chord} \quad A_v = 2 \cdot h_0 \cdot t_0$$

$$N_{3,Rd} = 214,34 \text{ [kN]} \quad \text{Tension capacity} \quad N_{3,Rd} = f_{y0} \cdot A_v / [\sqrt{3} \cdot \sin(\theta_3)] / \gamma_{M5}$$

$$\text{Final resistance:} \quad N_{3,Rd(INT)} = [(N_{3,Rd(\beta=0.85)} - \min[N_{3,Rd}; N_{3,Rd(\beta=1.0)}]) \cdot (1.0 - \beta)] / 0.15 + N_{3,Rd(\beta=1.0)}$$

$$N_{3,Rd(INT)} = 467,85 \text{ [kN]} \quad \text{Tension capacity}$$

$$|N_3| \leq N_{3,Rd(INT)} \quad |2,50| < 467,85 \quad \text{verified} \quad (0,01)$$

VERIFICATION OF WELDS**POST**

$\beta_w = 0,85$ Correlation coefficient [Table 4.1]

$\gamma_{M2} = 1,25$ Partial safety factor [Table 2.1]

Longitudinal weld

$\sigma_{\perp} = 161,16$ [MPa] Normal stress in a weld

$\tau_{\perp} = 161,16$ [MPa] Perpendicular tangent stress

$\tau_{\parallel} = 0,00$ [MPa] Tangent stress

$|\sigma_{\perp}| \leq 0.9 \cdot f_u / \gamma_{M2}$ $|161,16| < 309,60$ **verified** (0,52)

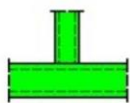
$\sqrt{[\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w \cdot \gamma_{M2})$ $322,32 < 404,71$ **verified** (0,80)

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

Ratio 0,80

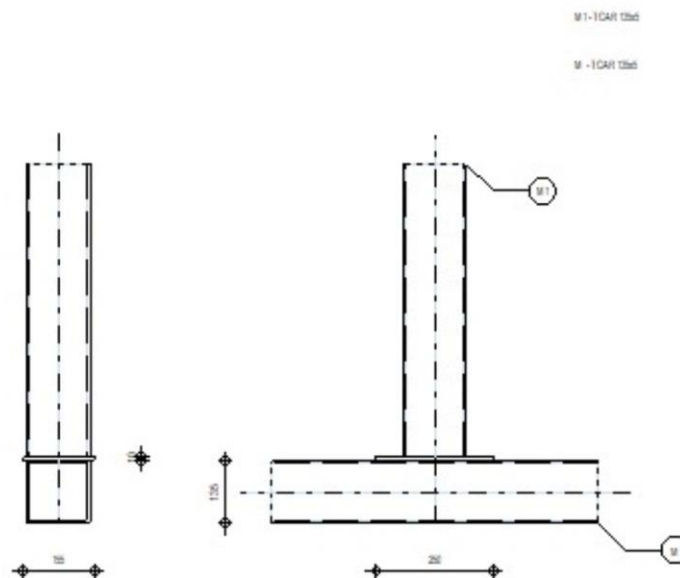


Autodesk Robot Structural Analysis Professional 2021

Design of truss node connection

EN 1993-1-8:2005/AC:2009

OK

Ratio
0,79**GENERAL**

Connection no.: 2

Connection name: Tube

Structure node: 132

Structure bars: 13, 11

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		13			11	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3880			3940	mm

HORIZONTAL BRACKETb_{ph} = 155 [mm] Widthl_{ph} = 250 [mm] Lengtht_{ph} = 10 [mm] Thickness

Material: Def

f_{yph} = 235,00 [MPa] Resistance**WELDS**a_d = 5 [mm] Thickness of welds of diagonals and postsa_{st} = 5 [mm] Thickness of stiffener welds**LOADS**

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORDN_{01,Ed} = -5,18 [kN] Axial forceM_{01,Ed} = -29,95 [kN*m] Bending momentN_{02,Ed} = -5,18 [kN] Axial forceM_{02,Ed} = -29,95 [kN*m] Bending moment**POST**

$N_3 = 1,29$ [kN] Axial force

$M_3 = 6,85$ [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS

CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

$\beta = 0,87$ Coefficient taking account of geometry of connection bars

$\beta = b_3/b_{ph}$ [1.5 (6)]

$\gamma = 6,75$ Coefficient taking account of geometry of the chord

$\gamma = b_0/(2*t_{ph})$ [1.5 (6)]

$n = 1,07$ Coefficient taking account of stresses in the chord

$n_0 = \sigma_{0,Ed}/f_{y0}$

$k_n = 0,81$ Coefficient taking account of stresses in the chord

$k_n = 1.3 - 0.4*n_0/\beta$

CHORD PUNCHING

Post

$b_{e,p} = 100$ [mm] Effective width for punching shear

$b_{e,p} = (10*b_3)/(b_0/t_0)$

$N_{3,Rd} = 746,23$ [kN] Tension capacity

$N_{3,Rd} = f_{y0} * t_{ph} / (\sqrt{3} * \sin(\theta_3)) * [2 * h_3 / \sin(\theta_3) + 2 * b_{e,p}] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|1,29| < 746,23$ **verified** (0,00)

TUBE BRACE FAILURE

Post

$W_{pl,3} = 102,96$ [cm³] Plastic section modulus

$b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$

$N_{3,Rd} = 715,00$ [kN] Tension capacity

$N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|1,29| < 715,00$ **verified** (0,00)

$M_{3,Rd} = 13,12$ [kN*m] Bending resistance

$M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$

$|M_3| \leq M_{3,Rd}$ $|6,85| < 13,12$ **verified** (0,52)

$$N_{3,Rd}/N_{3,Rd} + M_{3,Rd}/M_{3,Rd} \leq 1 \quad 0,52 < 1,00 \quad \text{verified} \quad (0,52)$$

CHORD SIDE WALL CRUSHING

POST

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 \cdot f_{y0} \cdot t_0 \cdot (h_3 + 5 \cdot t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |6,85| < 17,60 \quad \text{verified} \quad (0,39)$$

CHORD RESISTANCE

$$V_{pl,Rd} = 214,34 \text{ [kN]} \quad \text{Plastic resistance for shear} \quad V_{pl,Rd} = (A_v \cdot f_{y0}) / (\sqrt{3} \cdot \gamma_{M0})$$

$$|V_{Ed}| \leq V_{pl,Rd} \quad |0,00| < 214,34 \quad \text{verified} \quad (0,00)$$

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Chord resistance} \quad N_{0,Rd} = (A_0 \cdot f_{y0}) / \gamma_{M5}$$

$$|N_{02}| \leq N_{0,Rd} \quad |-5,18| < 690,25 \quad \text{verified} \quad (0,01)$$

INTERPOLATION

POST

Tube chord face failure

$$N_{3,Rd(\beta=0.85)} = 412,71 \text{ [kN]} \quad \text{Tension capacity} \quad N_{3,Rd(\beta=0.85)} = [(k_n \cdot f_{yp} \cdot t_{ph}^2) / (1-\beta) \cdot \sin(\theta_3)] \cdot [2 \cdot \beta / \sin(\theta_3) + 4 \cdot \sqrt{(1-\beta)}] / \gamma_M$$

Chord side wall buckling

$$f_b = 149,06 \text{ [MPa]} \quad \text{Buckling strength of the chord side wall} \quad f_b = \chi \cdot f_{y0}$$

$$N_{3,Rd(\beta=1.0)} = 193,37 \text{ [kN]} \quad \text{Tension capacity} \quad N_{3,Rd(\beta=1.0)} = [(k_n \cdot f_b \cdot t_0) / \sin(\theta_3)] \cdot [(2 \cdot h_3) / \sin(\theta_3) + 10 \cdot t_0] / \gamma_{M5}$$

Chord shear

$$A_v = 13,50 \text{ [cm}^2\text{]} \quad \text{Shear area of the chord} \quad A_v = 2 \cdot h_0 \cdot t_0$$

$$N_{3,Rd} = 214,34 \text{ [kN]} \quad \text{Tension capacity} \quad N_{3,Rd} = f_{y0} \cdot A_v / [\sqrt{3} \cdot \sin(\theta_3)] / \gamma_{M5}$$

$$\text{Final resistance:} \quad N_{3,Rd(INT)} = [(N_{3,Rd(\beta=0.85)} - \min[N_{3,Rd}; N_{3,Rd(\beta=1.0)}]) \cdot (1.0 - \beta)] / 0.15 + N_{3,Rd(\beta=1.0)}$$

$$N_{3,Rd(INT)} = 382,05 \text{ [kN]} \quad \text{Tension capacity}$$

$$|N_3| \leq N_{3,Rd(INT)} \quad |1,29| < 382,05 \quad \text{verified} \quad (0,00)$$

VERIFICATION OF WELDS**POST**

$\beta_w = 0,85$ Correlation coefficient [Table 4.1]

$\gamma_{M2} = 1,25$ Partial safety factor [Table 2.1]

Longitudinal weld

$\sigma_{\perp} = 160,07$ [MPa] Normal stress in a weld

$\tau_{\perp} = 160,07$ [MPa] Perpendicular tangent stress

$\tau_{\parallel} = 0,00$ [MPa] Tangent stress

$|\sigma_{\perp}| \leq 0.9 \cdot f_u / \gamma_{M2}$ $|160,07| < 309,60$ **verified** (0,52)

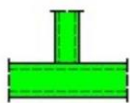
$\sqrt{[\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w \cdot \gamma_{M2})$ $320,13 < 404,71$ **verified** (0,79)

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

Ratio 0,79



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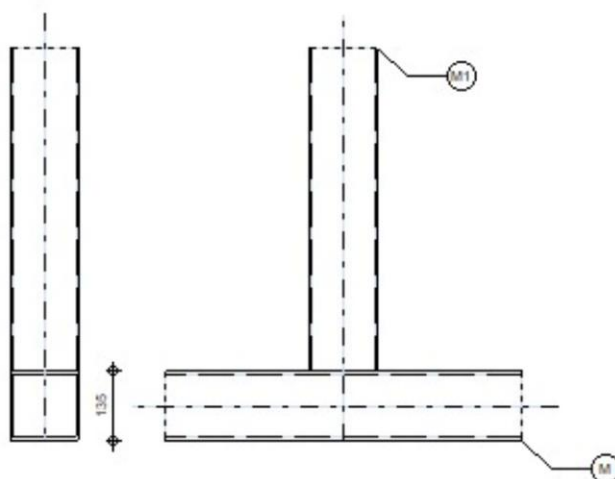
Design of truss node connection

EN 1993-1-8:2005/AC:2009

OKRatio
0,89

M1 - TCAR 135x5

M - TCAR 135x5

**GENERAL**

Connection no.: 39

Connection name: Column-Beam

Structure node: 2

Structure bars: 15, 1

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no.:		15			1	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3880			2400	mm

WELDS

a_d = 8 [mm] Thickness of welds of diagonals and posts

LOADS

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORD

N_{01,Ed} = -8,49 [kN] Axial force

M_{01,Ed} = 5,38 [kN*m] Bending moment

N_{02,Ed} = 0,00 [kN] Axial force

M_{02,Ed} = 0,00 [kN*m] Bending moment

POST

N₃ = -17,89 [kN] Axial force

M₃ = 11,14 [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS**CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005**

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

 $\beta = 1,00$ Coefficient taking account of geometry of connection bars $\beta = b_3/b_0$ [1.5 (6)]

 $\gamma = 13,50$ Coefficient taking account of geometry of the chord $\gamma = b_0/(2*t_0)$ [1.5 (6)]

TUBE BRACE FAILURE

Post

 $W_{pl,3} = 102,96$ [cm³] Plastic section modulus

 $b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$
 $N_{3,Rd} = 481,25$ [kN] Compression capacity $N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 481,25$ **verified** (0,04)

 $M_{3,Rd} = 13,12$ [kN*m] Bending resistance $M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$
 $|M_3| \leq M_{3,Rd}$ $|11,14| < 13,12$ **verified** (0,85)

 $N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$ $0,89 < 1,00$ **verified** (0,89)

CHORD SIDE WALL BUCKLING

Post

 $f_b = 149,06$ [MPa] Buckling strength of the chord side wall $f_b = \chi * f_{y0}$
 $N_{3,Rd} = 238,50$ [kN] Compression capacity $N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 238,50$ **verified** (0,07)

CHORD SHEAR

Post

 $A_v = 13,50$ [cm²] Shear area of the chord $A_v = 2 * h_0 * t_0$
 $N_{3,Rd} = 214,34$ [kN] Compression capacity $N_{3,Rd} = f_{y0} * A_v / [\sqrt{3} * \sin(\theta_3)] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 214,34$ **verified** (0,08)

CHORD RESISTANCE

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Compression capacity} \quad N_{0,Rd} = (A_0 * f_{y0}) / \gamma_{M5}$$

$$|N_{01}| \leq N_{0,Rd} \quad |-8,49| < 690,25 \quad \text{verified} \quad (0,01)$$

CHORD SIDE WALL CRUSHING**POST**

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 * f_{y0} * t_0 * (h_3 + 5 * t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |11,14| < 17,60 \quad \text{verified} \quad (0,63)$$

VERIFICATION OF WELDS**POST**

$$\beta_w = 0,85 \quad \text{Correlation coefficient} \quad [\text{Table 4.1}]$$

$$\gamma_{M2} = 1,25 \quad \text{Partial safety factor} \quad [\text{Table 2.1}]$$

Longitudinal weld

$$\sigma_{\perp} = -167,92 \text{ [MPa]} \quad \text{Normal stress in a weld}$$

$$\tau_{\perp} = -167,92 \text{ [MPa]} \quad \text{Perpendicular tangent stress}$$

$$\tau_{\parallel} = -0,00 \text{ [MPa]} \quad \text{Tangent stress}$$

$$|\sigma_{\perp}| \leq 0.9 * f_u / \gamma_{M2} \quad |-167,92| < 309,60 \quad \text{verified} \quad (0,54)$$

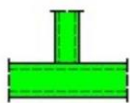
$$\sqrt{[\sigma_{\perp}^2 + 3 * (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w * \gamma_{M2}) \quad 335,84 < 404,71 \quad \text{verified} \quad (0,83)$$

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

Ratio 0,89



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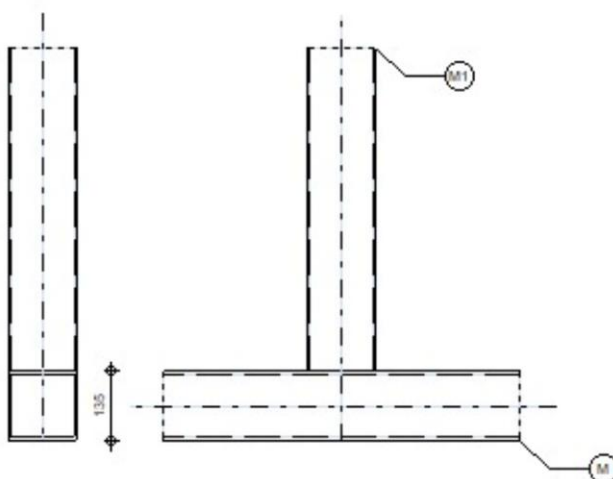
Design of truss node connection

EN 1993-1-8:2005/AC:2009

OKRatio
0,89

M1 - TCAR 135x5

M - TCAR 135x5

**GENERAL**

Connection no.: 40

Connection name: Column-Beam

Structure node: 2

Structure bars: 8, 1

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		8			1	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3940			2400	mm

WELDS

a_d = 8 [mm] Thickness of welds of diagonals and posts

LOADS

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORD

N_{01,Ed} = -6,00 [kN] Axial force

M_{01,Ed} = 3,90 [kN*m] Bending moment

N_{02,Ed} = 0,00 [kN] Axial force

M_{02,Ed} = 0,00 [kN*m] Bending moment

POST

N₃ = -17,89 [kN] Axial force

M₃ = 11,14 [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS**CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005**

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

 $\beta = 1,00$ Coefficient taking account of geometry of connection bars $\beta = b_3/b_0$ [1.5 (6)]

 $\gamma = 13,50$ Coefficient taking account of geometry of the chord $\gamma = b_0/(2*t_0)$ [1.5 (6)]

TUBE BRACE FAILURE

Post

 $W_{pl,3} = 102,96$ [cm³] Plastic section modulus

 $b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$
 $N_{3,Rd} = 481,25$ [kN] Compression capacity $N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 481,25$ **verified** (0,04)

 $M_{3,Rd} = 13,12$ [kN*m] Bending resistance $M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$
 $|M_3| \leq M_{3,Rd}$ $|11,14| < 13,12$ **verified** (0,85)

 $N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$ $0,89 < 1,00$ **verified** (0,89)

CHORD SIDE WALL BUCKLING

Post

 $f_b = 149,06$ [MPa] Buckling strength of the chord side wall $f_b = \chi * f_{y0}$
 $N_{3,Rd} = 238,50$ [kN] Compression capacity $N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 238,50$ **verified** (0,07)

CHORD SHEAR

Post

 $A_v = 13,50$ [cm²] Shear area of the chord $A_v = 2 * h_0 * t_0$
 $N_{3,Rd} = 214,34$ [kN] Compression capacity $N_{3,Rd} = f_{y0} * A_v / [\sqrt{3} * \sin(\theta_3)] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 214,34$ **verified** (0,08)

CHORD RESISTANCE

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Compression capacity} \quad N_{0,Rd} = (A_0 * f_{y0}) / \gamma_{M5}$$

$$|N_{01}| \leq N_{0,Rd} \quad |-6,00| < 690,25 \quad \text{verified} \quad (0,01)$$

CHORD SIDE WALL CRUSHING**POST**

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 * f_{y0} * t_0 * (h_3 + 5 * t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |11,14| < 17,60 \quad \text{verified} \quad (0,63)$$

VERIFICATION OF WELDS**POST**

$$\beta_w = 0,85 \quad \text{Correlation coefficient} \quad [\text{Table 4.1}]$$

$$\gamma_{M2} = 1,25 \quad \text{Partial safety factor} \quad [\text{Table 2.1}]$$

Longitudinal weld

$$\sigma_{\perp} = -167,92 \text{ [MPa]} \quad \text{Normal stress in a weld}$$

$$\tau_{\perp} = -167,92 \text{ [MPa]} \quad \text{Perpendicular tangent stress}$$

$$\tau_{\parallel} = -0,00 \text{ [MPa]} \quad \text{Tangent stress}$$

$$|\sigma_{\perp}| \leq 0.9 * f_u / \gamma_{M2} \quad |-167,92| < 309,60 \quad \text{verified} \quad (0,54)$$

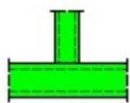
$$\sqrt{[\sigma_{\perp}^2 + 3 * (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w * \gamma_{M2}) \quad 335,84 < 404,71 \quad \text{verified} \quad (0,83)$$

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

Ratio 0,89



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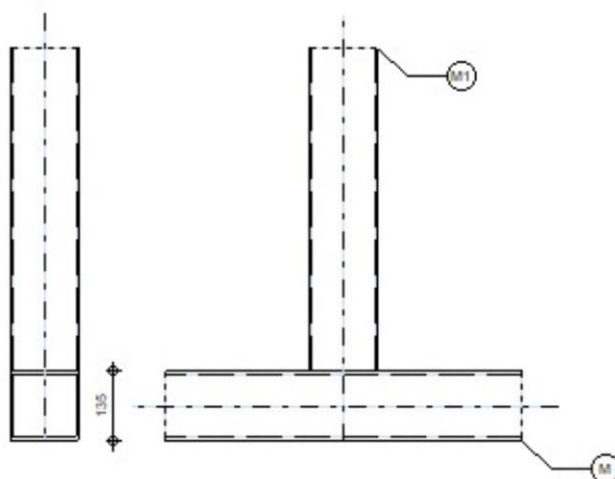
Design of truss node connection

EN 1993-1-8:2005/AC:2009

OKRatio
0,89

M1 - TCAR 135x5

M - TCAR 135x5

**GENERAL**

Connection no.: 41
Connection name: Column-Beam
Structure node: 4
Structure bars: 15, 2

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		15			2	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b_f	135			135	mm
	t_w	5			5	mm
	t_f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f_y	275,00			275,00	MPa
	f_u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3880			2400	mm

WELDS

$a_d = 8$ [mm] Thickness of welds of diagonals and posts

LOADS

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORD

$N_{01,Ed} = -8,49$ [kN] Axial force

$M_{01,Ed} = 5,38$ [kN*m] Bending moment

$N_{02,Ed} = 0,00$ [kN] Axial force

$M_{02,Ed} = 0,00$ [kN*m] Bending moment

POST

$N_3 = -17,89$ [kN] Axial force

$M_3 = -11,14$ [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS**CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005**

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

 $\beta = 1,00$ Coefficient taking account of geometry of connection bars $\beta = b_3/b_0$ [1.5 (6)]

 $\gamma = 13,50$ Coefficient taking account of geometry of the chord $\gamma = b_0/(2*t_0)$ [1.5 (6)]

TUBE BRACE FAILURE

Post

 $W_{pl,3} = 102,96$ [cm³] Plastic section modulus

 $b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$
 $N_{3,Rd} = 481,25$ [kN] Compression capacity $N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 481,25$ **verified** (0,04)

 $M_{3,Rd} = 13,12$ [kN*m] Bending resistance $M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$
 $|M_3| \leq M_{3,Rd}$ $|-11,14| < 13,12$ **verified** (0,85)

 $N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$ $0,89 < 1,00$ **verified** (0,89)

CHORD SIDE WALL BUCKLING

Post

 $f_b = 149,06$ [MPa] Buckling strength of the chord side wall $f_b = \chi * f_{y0}$
 $N_{3,Rd} = 238,50$ [kN] Compression capacity $N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 238,50$ **verified** (0,07)

CHORD SHEAR

Post

 $A_v = 13,50$ [cm²] Shear area of the chord $A_v = 2 * h_0 * t_0$
 $N_{3,Rd} = 214,34$ [kN] Compression capacity $N_{3,Rd} = f_{y0} * A_v / [\sqrt{3} * \sin(\theta_3)] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 214,34$ **verified** (0,08)

CHORD RESISTANCE

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Compression capacity} \quad N_{0,Rd} = (A_0 * f_{y0}) / \gamma_{M5}$$

$$|N_{01}| \leq N_{0,Rd} \quad |-8,49| < 690,25 \quad \text{verified} \quad (0,01)$$

CHORD SIDE WALL CRUSHING**POST**

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 * f_{y0} * t_0 * (h_3 + 5 * t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |-11,14| < 17,60 \quad \text{verified} \quad (0,63)$$

VERIFICATION OF WELDS**POST**

$$\beta_w = 0,85 \quad \text{Correlation coefficient} \quad [\text{Table 4.1}]$$

$$\gamma_{M2} = 1,25 \quad \text{Partial safety factor} \quad [\text{Table 2.1}]$$

Longitudinal weld

$$\sigma_{\perp} = 156,21 \text{ [MPa]} \quad \text{Normal stress in a weld}$$

$$\tau_{\perp} = 156,21 \text{ [MPa]} \quad \text{Perpendicular tangent stress}$$

$$\tau_{\parallel} = -0,00 \text{ [MPa]} \quad \text{Tangent stress}$$

$$|\sigma_{\perp}| \leq 0.9 * f_u / \gamma_{M2} \quad |156,21| < 309,60 \quad \text{verified} \quad (0,50)$$

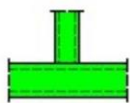
$$\sqrt{[\sigma_{\perp}^2 + 3 * (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w * \gamma_{M2}) \quad 312,42 < 404,71 \quad \text{verified} \quad (0,77)$$

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

Ratio 0,89



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Design of truss node connection

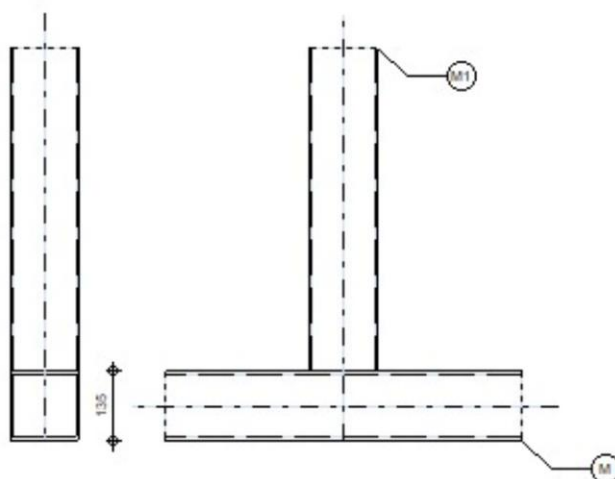
EN 1993-1-8:2005/AC:2009

OK

Ratio
0,89

M1 - TCAR 135x5

M - TCAR 135x5

**GENERAL**

Connection no.: 42

Connection name: Column-Beam

Structure node: 4

Structure bars: 10, 2

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no.:		10			2	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b_f	135			135	mm
	t_w	5			5	mm
	t_f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f_y	275,00			275,00	MPa
	f_u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3940			2400	mm

WELDS

$a_d = 8$ [mm] Thickness of welds of diagonals and posts

LOADS

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORD

$N_{01,Ed} = -6,00$ [kN] Axial force

$M_{01,Ed} = 3,90$ [kN*m] Bending moment

$N_{02,Ed} = 0,00$ [kN] Axial force

$M_{02,Ed} = 0,00$ [kN*m] Bending moment

POST

$N_3 = -17,89$ [kN] Axial force

$M_3 = -11,14$ [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS**CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005**

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

 $\beta = 1,00$ Coefficient taking account of geometry of connection bars $\beta = b_3/b_0$ [1.5 (6)]

 $\gamma = 13,50$ Coefficient taking account of geometry of the chord $\gamma = b_0/(2*t_0)$ [1.5 (6)]

TUBE BRACE FAILURE

Post

 $W_{pl,3} = 102,96$ [cm³] Plastic section modulus

 $b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$
 $N_{3,Rd} = 481,25$ [kN] Compression capacity $N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 481,25$ **verified** (0,04)

 $M_{3,Rd} = 13,12$ [kN*m] Bending resistance $M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$
 $|M_3| \leq M_{3,Rd}$ $|-11,14| < 13,12$ **verified** (0,85)

 $N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$ $0,89 < 1,00$ **verified** (0,89)

CHORD SIDE WALL BUCKLING

Post

 $f_b = 149,06$ [MPa] Buckling strength of the chord side wall $f_b = \chi * f_{y0}$
 $N_{3,Rd} = 238,50$ [kN] Compression capacity $N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 238,50$ **verified** (0,07)

CHORD SHEAR

Post

 $A_v = 13,50$ [cm²] Shear area of the chord $A_v = 2 * h_0 * t_0$
 $N_{3,Rd} = 214,34$ [kN] Compression capacity $N_{3,Rd} = f_{y0} * A_v / [\sqrt{3} * \sin(\theta_3)] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-17,89| < 214,34$ **verified** (0,08)

CHORD RESISTANCE

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Compression capacity} \quad N_{0,Rd} = (A_0 * f_{y0}) / \gamma_{M5}$$

$$|N_{01}| \leq N_{0,Rd} \quad |-6,00| < 690,25 \quad \text{verified} \quad (0,01)$$

CHORD SIDE WALL CRUSHING**POST**

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 * f_{y0} * t_0 * (h_3 + 5 * t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |-11,14| < 17,60 \quad \text{verified} \quad (0,63)$$

VERIFICATION OF WELDS**POST**

$$\beta_w = 0,85 \quad \text{Correlation coefficient} \quad [\text{Table 4.1}]$$

$$\gamma_{M2} = 1,25 \quad \text{Partial safety factor} \quad [\text{Table 2.1}]$$

Longitudinal weld

$$\sigma_{\perp} = 156,21 \text{ [MPa]} \quad \text{Normal stress in a weld}$$

$$\tau_{\perp} = 156,21 \text{ [MPa]} \quad \text{Perpendicular tangent stress}$$

$$\tau_{\parallel} = -0,00 \text{ [MPa]} \quad \text{Tangent stress}$$

$$|\sigma_{\perp}| \leq 0.9 * f_u / \gamma_{M2} \quad |156,21| < 309,60 \quad \text{verified} \quad (0,50)$$

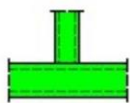
$$\sqrt{[\sigma_{\perp}^2 + 3 * (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w * \gamma_{M2}) \quad 312,42 < 404,71 \quad \text{verified} \quad (0,77)$$

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

Ratio 0,89



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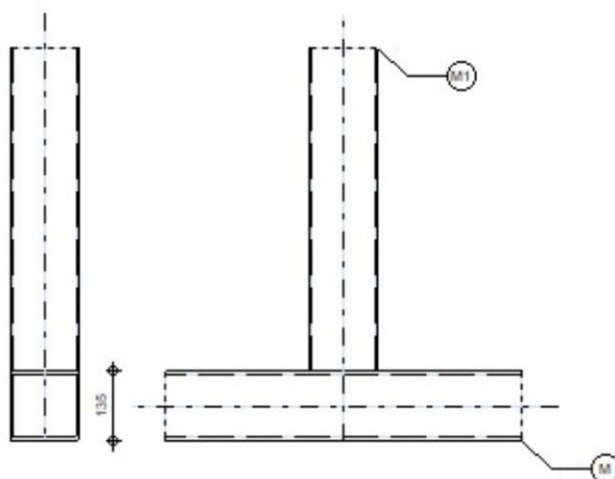
Design of truss node connection

EN 1993-1-8:2005/AC:2009

OKRatio
0,91

M1 - TCAR 135x5

M - TCAR 135x5

**GENERAL**

Connection no.: 43

Connection name: Column-Beam

Structure node: 8

Structure bars: 14, 4

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		14			4	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3880			2400	mm

WELDS

a_d = 8 [mm] Thickness of welds of diagonals and posts

LOADS

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORD

N_{01,Ed} = -6,63 [kN] Axial force

M_{01,Ed} = 5,81 [kN*m] Bending moment

N_{02,Ed} = 0,00 [kN] Axial force

M_{02,Ed} = 0,00 [kN*m] Bending moment

POST

N₃ = -18,95 [kN] Axial force

M₃ = -11,47 [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS**CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005**

$\gamma_{M5} = 1,00$ Partial safety factor [Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

$\beta = 1,00$ Coefficient taking account of geometry of connection bars $\beta = b_3/b_0$ [1.5 (6)]
 $\gamma = 13,50$ Coefficient taking account of geometry of the chord $\gamma = b_0/(2*t_0)$ [1.5 (6)]

TUBE BRACE FAILURE

Post

$W_{pl,3} = 102,96$ [cm³] Plastic section modulus

$b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$

$N_{3,Rd} = 481,25$ [kN] Compression capacity $N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-18,95| < 481,25$ **verified** (0,04)

$M_{3,Rd} = 13,12$ [kN*m] Bending resistance $M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$

$|M_3| \leq M_{3,Rd}$ $|-11,47| < 13,12$ **verified** (0,87)

$N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$ $0,91 < 1,00$ **verified** (0,91)

CHORD SIDE WALL BUCKLING

Post

$f_b = 149,06$ [MPa] Buckling strength of the chord side wall $f_b = \chi * f_{y0}$

$N_{3,Rd} = 238,50$ [kN] Compression capacity $N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-18,95| < 238,50$ **verified** (0,08)

CHORD SHEAR

Post

$A_v = 13,50$ [cm²] Shear area of the chord $A_v = 2 * h_0 * t_0$

$N_{3,Rd} = 214,34$ [kN] Compression capacity $N_{3,Rd} = f_{y0} * A_v / [\sqrt{3} * \sin(\theta_3)] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-18,95| < 214,34$ **verified** (0,09)

CHORD RESISTANCE

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Compression capacity} \quad N_{0,Rd} = (A_0 * f_{y0}) / \gamma_{M5}$$

$$|N_{01}| \leq N_{0,Rd} \quad |-6,63| < 690,25 \quad \text{verified} \quad (0,01)$$

CHORD SIDE WALL CRUSHING

POST

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 * f_{y0} * t_0 * (h_3 + 5 * t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |-11,47| < 17,60 \quad \text{verified} \quad (0,65)$$

VERIFICATION OF WELDS

POST

$$\beta_w = 0,85 \quad \text{Correlation coefficient} \quad [\text{Table 4.1}]$$

$$\gamma_{M2} = 1,25 \quad \text{Partial safety factor} \quad [\text{Table 2.1}]$$

Longitudinal weld

$$\sigma_{\perp} = 160,65 \text{ [MPa]} \quad \text{Normal stress in a weld}$$

$$\tau_{\perp} = 160,65 \text{ [MPa]} \quad \text{Perpendicular tangent stress}$$

$$\tau_{\parallel} = -0,00 \text{ [MPa]} \quad \text{Tangent stress}$$

$$|\sigma_{\perp}| \leq 0.9 * f_u / \gamma_{M2} \quad |160,65| < 309,60 \quad \text{verified} \quad (0,52)$$

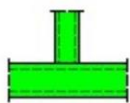
$$\sqrt{[\sigma_{\perp}^2 + 3 * (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w * \gamma_{M2}) \quad 321,30 < 404,71 \quad \text{verified} \quad (0,79)$$

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

Ratio 0,91



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Design of truss node connection

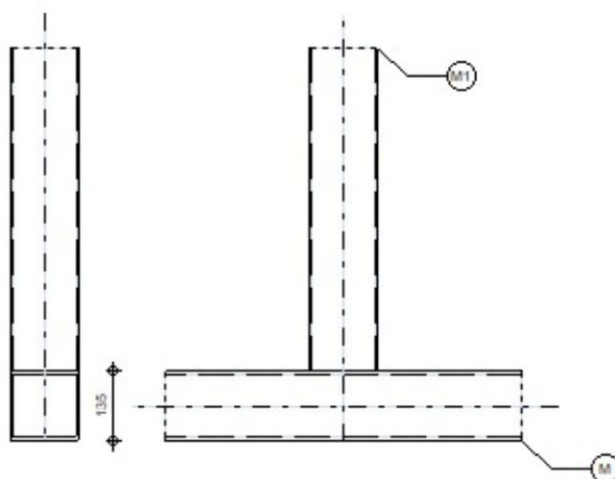
EN 1993-1-8:2005/AC:2009

OK

Ratio
0,91

M1 - TCAR 135x5

M - TCAR 135x5

**GENERAL**

Connection no.: 44

Connection name: Column-Beam

Structure node: 8

Structure bars: 9, 4

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		9			4	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3940			2400	mm

WELDS

a_d = 8 [mm] Thickness of welds of diagonals and posts

LOADS

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORD

N_{01,Ed} = -3,61 [kN] Axial force

M_{01,Ed} = 3,88 [kN*m] Bending moment

N_{02,Ed} = 0,00 [kN] Axial force

M_{02,Ed} = 0,00 [kN*m] Bending moment

POST

N₃ = -18,95 [kN] Axial force

M₃ = -11,47 [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS**CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005**

$\gamma_{M5} = 1,00$ Partial safety factor

[Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

 $\beta = 1,00$ Coefficient taking account of geometry of connection bars $\beta = b_3/b_0$ [1.5 (6)]

 $\gamma = 13,50$ Coefficient taking account of geometry of the chord $\gamma = b_0/(2*t_0)$ [1.5 (6)]

TUBE BRACE FAILURE

Post

 $W_{pl,3} = 102,96$ [cm³] Plastic section modulus

 $b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$
 $N_{3,Rd} = 481,25$ [kN] Compression capacity $N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-18,95| < 481,25$ **verified** (0,04)

 $M_{3,Rd} = 13,12$ [kN*m] Bending resistance $M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$
 $|M_3| \leq M_{3,Rd}$ $|-11,47| < 13,12$ **verified** (0,87)

 $N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$ $0,91 < 1,00$ **verified** (0,91)

CHORD SIDE WALL BUCKLING

Post

 $f_b = 149,06$ [MPa] Buckling strength of the chord side wall $f_b = \chi * f_{y0}$
 $N_{3,Rd} = 238,50$ [kN] Compression capacity $N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-18,95| < 238,50$ **verified** (0,08)

CHORD SHEAR

Post

 $A_v = 13,50$ [cm²] Shear area of the chord $A_v = 2 * h_0 * t_0$
 $N_{3,Rd} = 214,34$ [kN] Compression capacity $N_{3,Rd} = f_{y0} * A_v / [\sqrt{3} * \sin(\theta_3)] / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-18,95| < 214,34$ **verified** (0,09)

CHORD RESISTANCE

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Compression capacity} \quad N_{0,Rd} = (A_0 * f_{y0}) / \gamma_{M5}$$

$$|N_{01}| \leq N_{0,Rd} \quad |-3,61| < 690,25 \quad \text{verified} \quad (0,01)$$

CHORD SIDE WALL CRUSHING**POST**

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 * f_{y0} * t_0 * (h_3 + 5 * t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |-11,47| < 17,60 \quad \text{verified} \quad (0,65)$$

VERIFICATION OF WELDS**POST**

$$\beta_w = 0,85 \quad \text{Correlation coefficient} \quad [\text{Table 4.1}]$$

$$\gamma_{M2} = 1,25 \quad \text{Partial safety factor} \quad [\text{Table 2.1}]$$

Longitudinal weld

$$\sigma_{\perp} = 160,65 \text{ [MPa]} \quad \text{Normal stress in a weld}$$

$$\tau_{\perp} = 160,65 \text{ [MPa]} \quad \text{Perpendicular tangent stress}$$

$$\tau_{\parallel} = -0,00 \text{ [MPa]} \quad \text{Tangent stress}$$

$$|\sigma_{\perp}| \leq 0.9 * f_u / \gamma_{M2} \quad |160,65| < 309,60 \quad \text{verified} \quad (0,52)$$

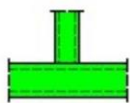
$$\sqrt{[\sigma_{\perp}^2 + 3 * (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w * \gamma_{M2}) \quad 321,30 < 404,71 \quad \text{verified} \quad (0,79)$$

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

Ratio 0,91



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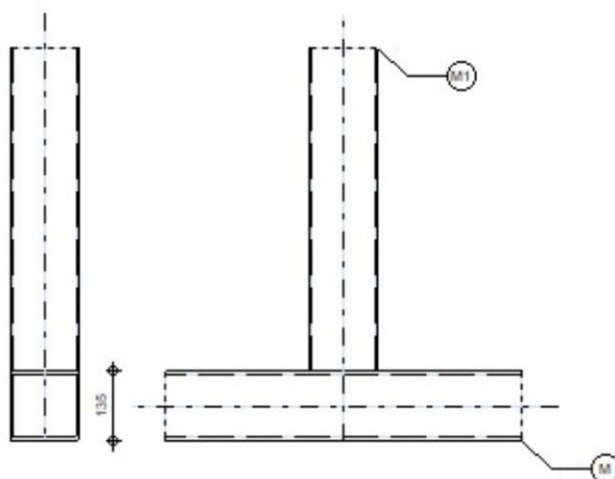
Design of truss node connection

EN 1993-1-8:2005/AC:2009

OKRatio
0,91

M1 - TCAR 135x5

M - TCAR 135x5

**GENERAL**

Connection no.: 45

Connection name: Column-Beam

Structure node: 6

Structure bars: 14, 3

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		14			3	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3880			2400	mm

WELDS

a_d = 8 [mm] Thickness of welds of diagonals and posts

LOADS

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORD

N_{01,Ed} = -6,63 [kN] Axial force

M_{01,Ed} = 5,81 [kN*m] Bending moment

N_{02,Ed} = 0,00 [kN] Axial force

M_{02,Ed} = 0,00 [kN*m] Bending moment

POST

N₃ = -18,95 [kN] Axial force

M₃ = 11,47 [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS**CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005**

$\gamma_{M5} = 1,00$ Partial safety factor [Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

$\beta = 1,00$ Coefficient taking account of geometry of connection bars $\beta = b_3/b_0$ [1.5 (6)]
 $\gamma = 13,50$ Coefficient taking account of geometry of the chord $\gamma = b_0/(2*t_0)$ [1.5 (6)]

TUBE BRACE FAILURE

Post

$W_{pl,3} = 102,96$ [cm³] Plastic section modulus

$b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$

$N_{3,Rd} = 481,25$ [kN] Compression capacity $N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-18,95| < 481,25$ **verified** (0,04)

$M_{3,Rd} = 13,12$ [kN*m] Bending resistance $M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$

$|M_3| \leq M_{3,Rd}$ $|11,47| < 13,12$ **verified** (0,87)

$N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$ $0,91 < 1,00$ **verified** (0,91)

CHORD SIDE WALL BUCKLING

Post

$f_b = 149,06$ [MPa] Buckling strength of the chord side wall $f_b = \chi * f_{y0}$

$N_{3,Rd} = 238,50$ [kN] Compression capacity $N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-18,95| < 238,50$ **verified** (0,08)

CHORD SHEAR

Post

$A_v = 13,50$ [cm²] Shear area of the chord $A_v = 2 * h_0 * t_0$

$N_{3,Rd} = 214,34$ [kN] Compression capacity $N_{3,Rd} = f_{y0} * A_v / [\sqrt{3} * \sin(\theta_3)] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-18,95| < 214,34$ **verified** (0,09)

CHORD RESISTANCE

$$N_{0,Rd} = 690,25 \text{ [kN]} \quad \text{Compression capacity} \quad N_{0,Rd} = (A_0 * f_{y0}) / \gamma_{M5}$$

$$|N_{01}| \leq N_{0,Rd} \quad |-6,63| < 690,25 \quad \text{verified} \quad (0,01)$$

CHORD SIDE WALL CRUSHING**POST**

$$M_{3,Rd} = 17,60 \text{ [kN*m]} \quad \text{Bending resistance} \quad M_{3,Rd} = 0.5 * f_{y0} * t_0 * (h_3 + 5 * t_0)^2 / \gamma_{M5}$$

$$|M_3| \leq M_{3,Rd} \quad |11,47| < 17,60 \quad \text{verified} \quad (0,65)$$

VERIFICATION OF WELDS**POST**

$$\beta_w = 0,85 \quad \text{Correlation coefficient} \quad [\text{Table 4.1}]$$

$$\gamma_{M2} = 1,25 \quad \text{Partial safety factor} \quad [\text{Table 2.1}]$$

Longitudinal weld

$$\sigma_{\perp} = -173,06 \text{ [MPa]} \quad \text{Normal stress in a weld}$$

$$\tau_{\perp} = -173,06 \text{ [MPa]} \quad \text{Perpendicular tangent stress}$$

$$\tau_{\parallel} = -0,00 \text{ [MPa]} \quad \text{Tangent stress}$$

$$|\sigma_{\perp}| \leq 0.9 * f_u / \gamma_{M2} \quad |-173,06| < 309,60 \quad \text{verified} \quad (0,56)$$

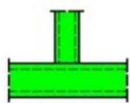
$$\sqrt{[\sigma_{\perp}^2 + 3 * (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w * \gamma_{M2}) \quad 346,12 < 404,71 \quad \text{verified} \quad (0,86)$$

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Connection conforms to the code

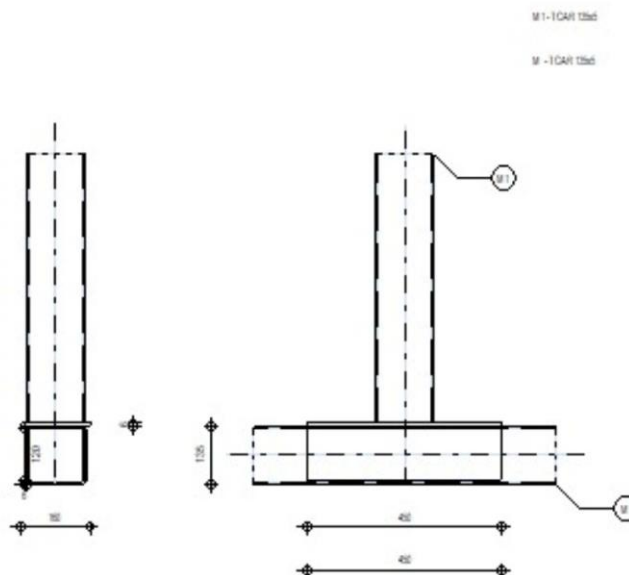
Ratio 0,91



Autodesk Robot Structural Analysis Professional 2021

Design of truss node connection

EN 1993-1-8:2005/AC:2009

Ratio
0,91**GENERAL**

Connection no.: 46

Connection name: Column-Beam

Structure node: 6

Structure bars: 7, 3

GEOMETRY**BARS**

		Chord	Diagonal 1	Diagonal 2	Post	
Bar no. :		7			3	
Section:		TCAR 135x5			TCAR 135x5	
	h	135			135	mm
	b _f	135			135	mm
	t _w	5			5	mm
	t _f	5			5	mm
	r	11			11	mm
Material:		S275			S275	
	f _y	275,00			275,00	MPa
	f _u	430,00			430,00	MPa
Angle	θ	0,0			90,0	Deg
Length	l	3940			2400	mm

VERTICAL BRACKETb_{pv} = 120 [mm] Widthl_{pv} = 450 [mm] Lengtht_{pv} = 6 [mm] Thickness

Material: Def

f_{ypv} = 235,00 [MPa] Resistance**HORIZONTAL BRACKET**b_{ph} = 160 [mm] Widthl_{ph} = 450 [mm] Lengtht_{ph} = 6 [mm] Thickness

Material: Def

f_{ypv} = 235,00 [MPa] Resistance**WELDS**a_d = 8 [mm] Thickness of welds of diagonals and postsa_{st} = 5 [mm] Thickness of stiffener welds**LOADS**

Case: 66: ULS 31 (1+2)*1.35+3*0.90+59*1.50

CHORD

$N_{01,Ed} = -3,61$ [kN] Axial force
 $M_{01,Ed} = 3,88$ [kN*m] Bending moment

$N_{02,Ed} = 0,00$ [kN] Axial force
 $M_{02,Ed} = 0,00$ [kN*m] Bending moment

POST

$N_3 = -18,95$ [kN] Axial force
 $M_3 = 11,47$ [kN*m] Bending moment

Shear forces were not included in the connection verification. The connection was designed as a truss node.

RESULTS

CAPACITY VERIFICATION EUROCODE 3: EN 1993-1-8:2005

$\gamma_{M5} = 1,00$ Partial safety factor [Table 2.1]

The connection geometry does not comply with the table. [7.9]

FAILURE MODES FOR JOINTS (RHS CHORD MEMBERS) [Table 7.11] for $N_{i,Rd}$ and [Table 7.14] for $M_{i,Rd}$

GEOMETRICAL PARAMETERS

$\beta = 1,00$ Coefficient taking account of geometry of connection bars $\beta = b_3/b_0$ [1.5 (6)]
 $\gamma = 11,25$ Coefficient taking account of geometry of the chord $\gamma = b_0/(2*t_{ph})$ [1.5 (6)]

TUBE BRACE FAILURE

Post

$W_{pl,3} = 102,96$ [cm³] Plastic section modulus

$b_{eff} = 50$ [mm] Effective width in the connection of the post to the chord $b_{eff} = [10/(b_0/t_0)] * [(f_{y0} * t_0)/(f_{y3} * t_3)] * b$

$N_{3,Rd} = 541,75$ [kN] Compression capacity $N_{3,Rd} = f_{y3} * t_3 * (2 * h_3 - 4 * t_3 + 2 * b_{eff}) / \gamma_{M5}$
 $|N_3| \leq N_{3,Rd}$ $|-18,95| < 541,75$ **verified** (0,03)

$M_{3,Rd} = 13,12$ [kN*m] Bending resistance $M_{3,Rd} = f_{y3} * (W_{pl,3} - (1 - b_{eff}/b_3) * b_3 * (h_3 - t_3) * t_3) / \gamma_{M5}$
 $|M_3| \leq M_{3,Rd}$ $|11,47| < 13,12$ **verified** (0,87)

$N_3/N_{3,Rd} + M_3/M_{3,Rd} \leq 1$ $0,91 < 1,00$ **verified** (0,91)

CHORD SIDE WALL BUCKLING

Post

$f_b = 149,06$ [MPa] Buckling strength of the chord side wall $f_b = \chi * f_{y0}$

$N_{3,Rd} = 238,50$ [kN] Compression capacity $N_{3,Rd} = [(k_n * f_b * t_0) / \sin(\theta_3)] * [(2 * h_3) / \sin(\theta_3) + 10 * t_0] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-18,95| < 238,50$ **verified** (0,08)

CHORD SHEAR

Post

$A_v = 29,70$ [cm²] Shear area of the chord $A_v = 2 * h_0 * (t_0 + t_{pv})$

$N_{3,Rd} = 471,55$ [kN] Compression capacity $N_{3,Rd} = f_{y0} * A_v * [\sqrt{3} * \sin(\theta_3)] / \gamma_{M5}$

$|N_3| \leq N_{3,Rd}$ $|-18,95| < 471,55$ **verified** (0,04)

CHORD RESISTANCE

$N_{0,Rd} = 816,75$ [kN] Compression capacity $N_{0,Rd} = (A_0 * f_{y0}) / \gamma_{M5}$

$|N_{01}| \leq N_{0,Rd}$ $|-3,61| < 816,75$ **verified** (0,00)

CHORD SIDE WALL CRUSHING

Post

$M_{3,Rd} = 17,60$ [kN*m] Bending resistance $M_{3,Rd} = 0.5 * f_{y0} * t_0 * (h_3 + 5 * t_0)^2 / \gamma_{M5}$

$|M_3| \leq M_{3,Rd}$ $|11,47| < 17,60$ **verified** (0,65)

VERIFICATION OF WELDS

Post

$\beta_w = 0,85$ Correlation coefficient [Table 4.1]

$\gamma_{M2} = 1,25$ Partial safety factor [Table 2.1]

Longitudinal weld

$\sigma_{\perp} = -173,06$ [MPa] Normal stress in a weld

$\tau_{\perp} = -173,06$ [MPa] Perpendicular tangent stress

$\tau_{\parallel} = -0,00$ [MPa] Tangent stress

$ \sigma_{\perp} \leq 0.9 \cdot f_u / \gamma_{M2}$	$ -173,06 < 309,60$	verified	(0,56)
$\sqrt{[\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w \cdot \gamma_{M2})$	$346,12 < 404,71$	verified	(0,86)

REMARKS

Ratio of the post width to the chord width is too large $1,00 > 0,85$

Thickness of the vertical stiffener is too small $6 \text{ [mm]} < 10 \text{ [mm]}$

Connection conforms to the code

Ratio 0,91

ΣΦΡΑΓΙΔΑ:

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