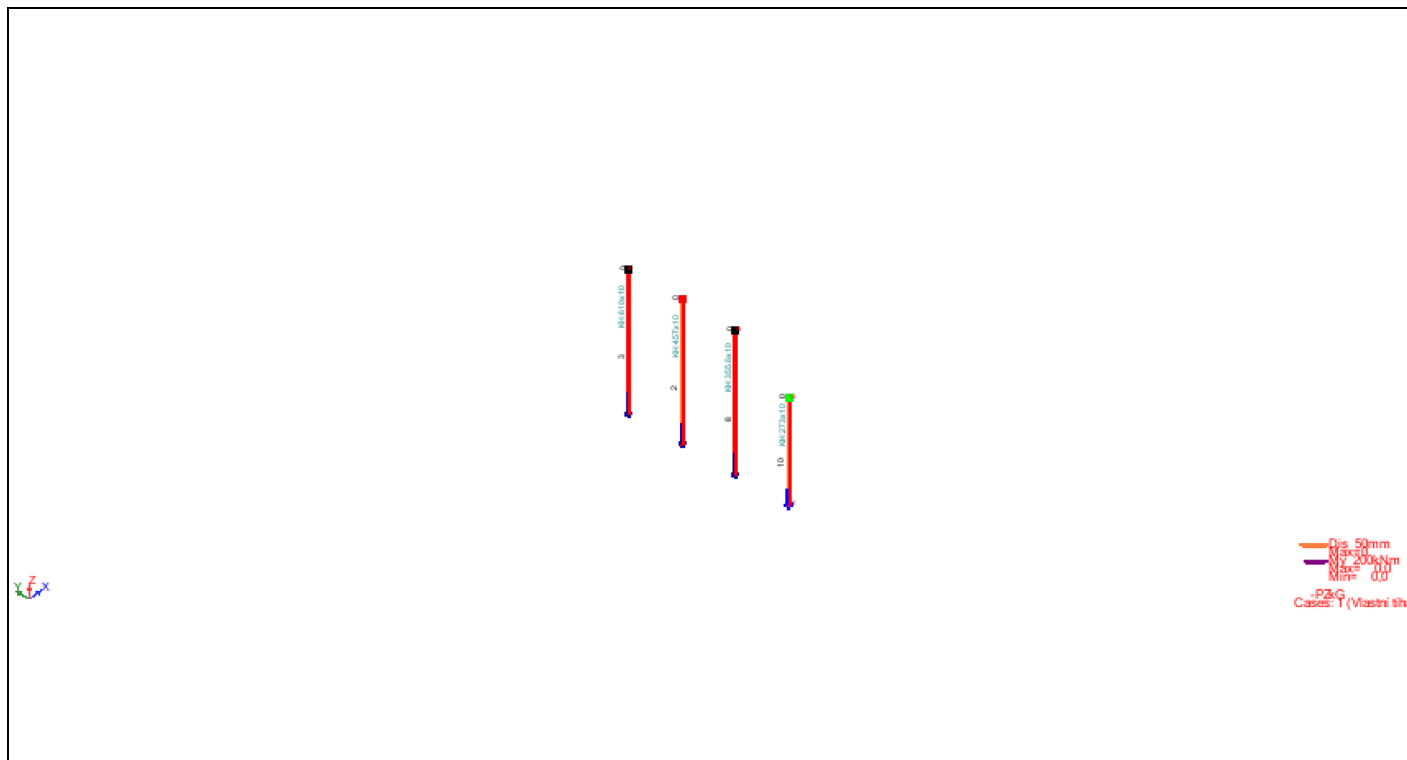


POSOUZENÍ OCELOVÝCH SLOUPŮ

Geometry



Data - Nodes

Node	X (m)	Y (m)	Z (m)	Support code	Support
3	79,00	0,40	0,0	xxxxxx	Fixed
5	79,00	3,40	0,0	xxxxxx	Fixed
6	79,00	3,40	8,00		
7	79,00	6,40	0,0	xxxxxx	Fixed
8	79,00	6,40	8,00		
11	79,00	-2,60	0,0	xxxxxx	Fixed
14	79,00	-2,60	6,00		
15	79,00	0,40	8,00		

Data - Bars

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
2	5	6	KH 457x10	S355	8,00	0,0	Konzola
3	7	8	KH 610x10	S355	8,00	0,0	Konzola
8	3	15	KH 355.6x10	S355	8,00	0,0	Konzola
10	11	14	KH 273x10	S355	6,00	0,0	Konzola

Data - Sections

Section name	Bar list	AX (mm2)	AY (mm2)	AZ (mm2)	IX (mm4)	IY (mm4)	IZ (mm4)
KH 355.6x10	8	10900	5450	5450	324470000	162230000	162230000
KH 610x10	3	18800	9400	9400	1696930000	848470000	848470000
KH 457x10	2	14000	7000	7000	701830000	350910000	350910000
KH 273x10	10	8260	4130	4130	143080000	71540000	71540000

Data - Materials

	Material	E (MPa)	G (MPa)	NI	LX (1/°C)	RO (kN/m3)	Re (MPa)
1	S355	205000,00	80000,00	0,30	0,00	77,01	355,00

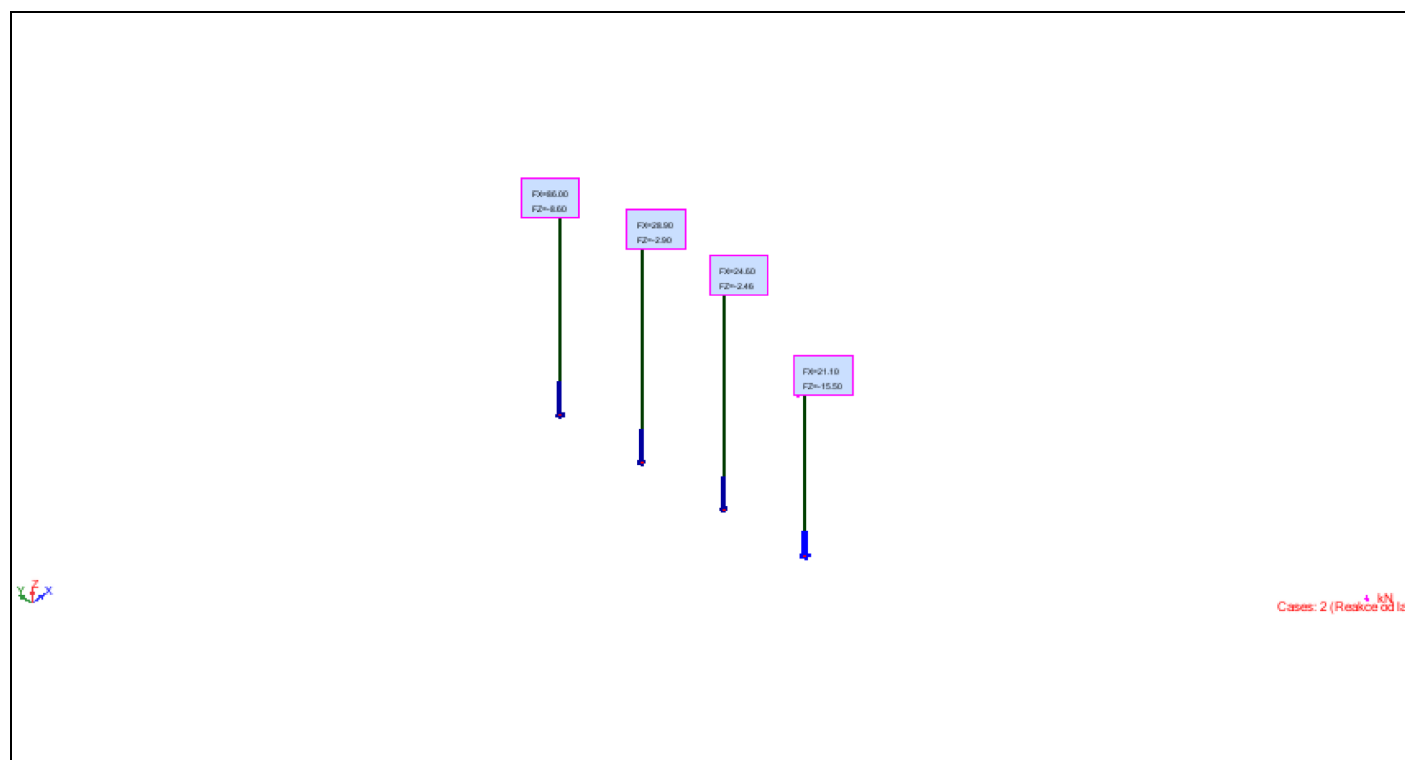
Data - Supports

Support name	List of nodes	List of edges	List of objects	Support conditions
Fixed	3 5 7 11			UX UY UZ RX RY RZ

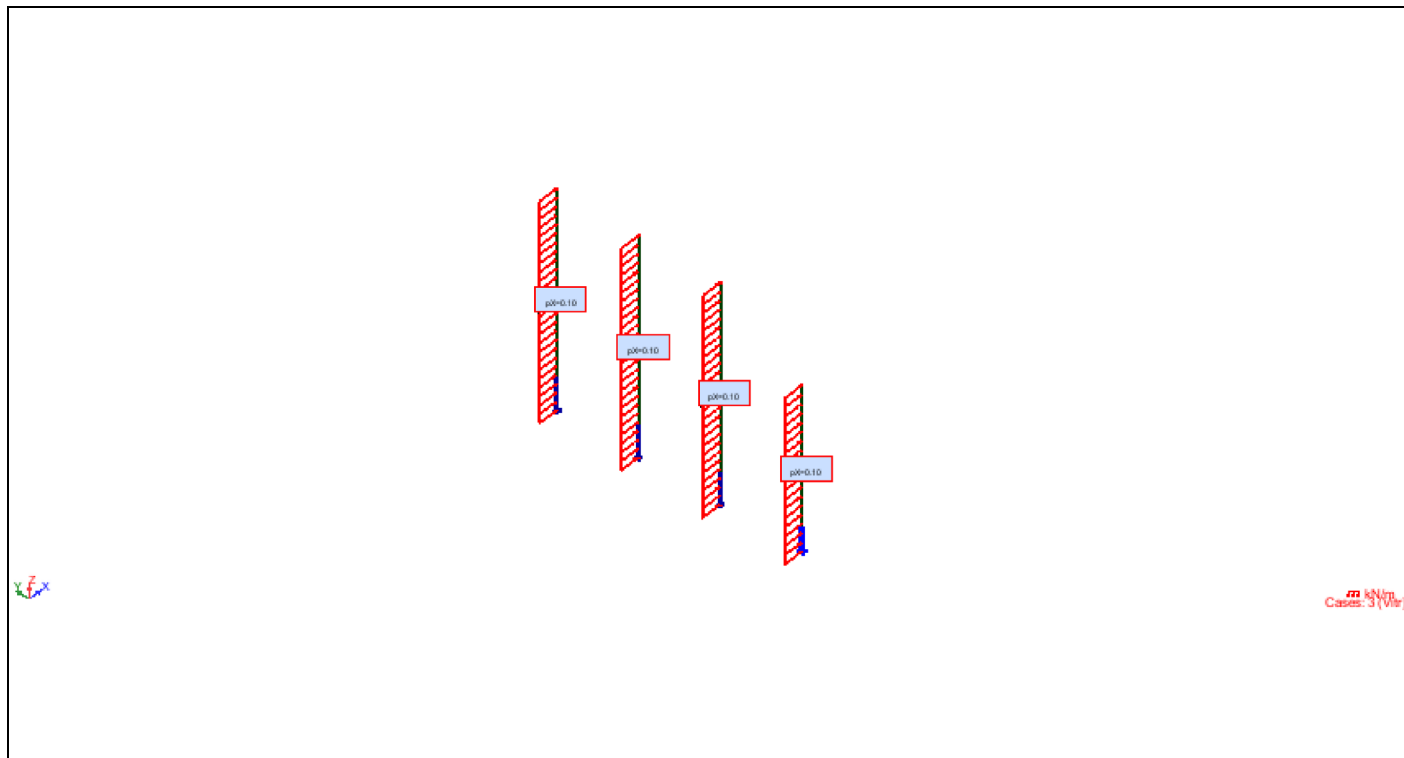
Data - Stories

Case/Story	Name	Mass (kg)	G (x,y,z) (m)	R (x,y,z) (m)	Ix (kgm2)	
Case/Story	Iy (kgm2)	Iz (kgm2)	ex0 (m)	ey0 (m)	ex2 (m)	ey2 (m)

Reakce od lana



(Vítr)



Loads - Values

Case	Load type	List	Load values
1	self-weight	2 3 8 10	PZ Negative Factor=1,00
2	Assembling:self-weight	2 3 8 10	PZ Negative Factor=1,00
2	nodal force		FX=24,60(kN) FZ=-2,50(kN)
2	nodal force	8	FX=86,00(kN) FZ=-8,60(kN)
2	nodal force	6	FX=28,90(kN) FZ=-2,90(kN)
2	nodal force	15	FX=24,60(kN) FZ=-2,46(kN)
2	nodal force	14	FX=21,10(kN) FZ=-15,50(kN)
3	uniform load	2 3 8 10	PX=0,10(kN/m)

Loads – Cases

Case	Label	Case name
1	DL1	Vlastní tíha
2	Pletivo	Reakce od lana
3	Vítr	Vítr
5		COMB1
6		COMB2
7		Nonlinear
8		Nonlinear

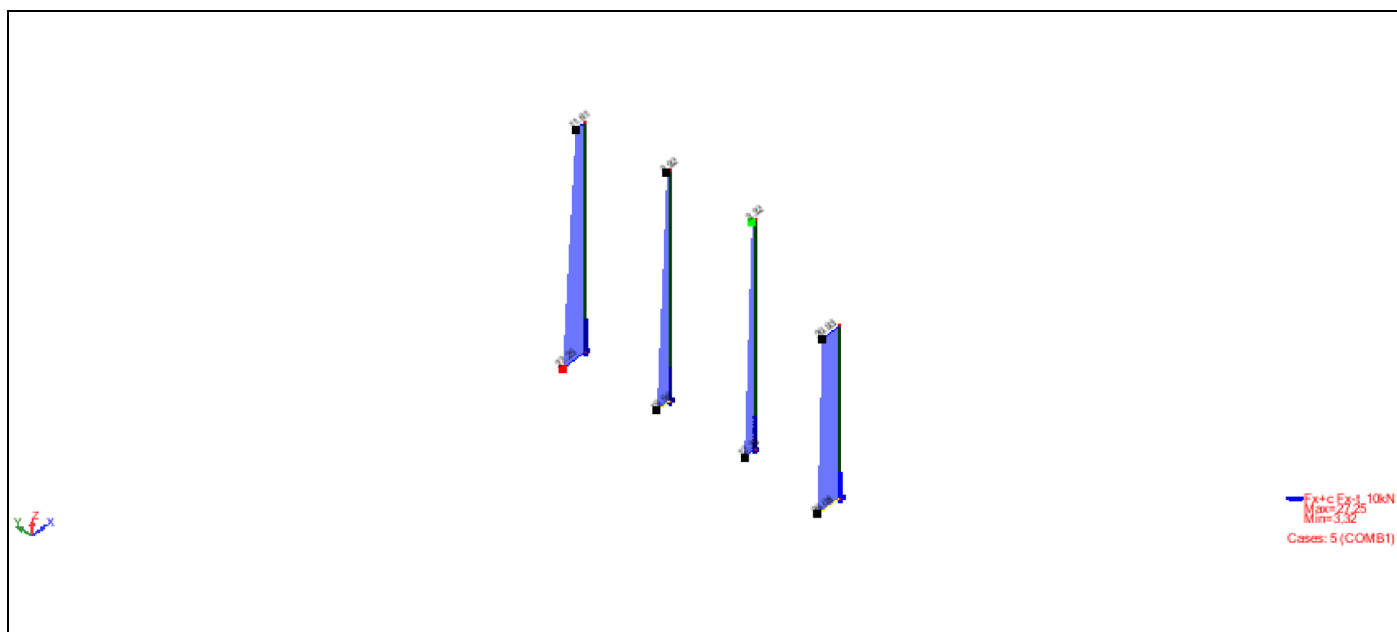
Case	Nature	Analysis type
1	Structural	Static - Linear
2	Non-structural	Static - Linear
3	wind	Static - Linear

Case	Nature	Analysis type
5	Structural	Linear Combination
6	Structural	Linear Combination
7	Structural	Combination NL PD
8	Structural	Combination NL PD

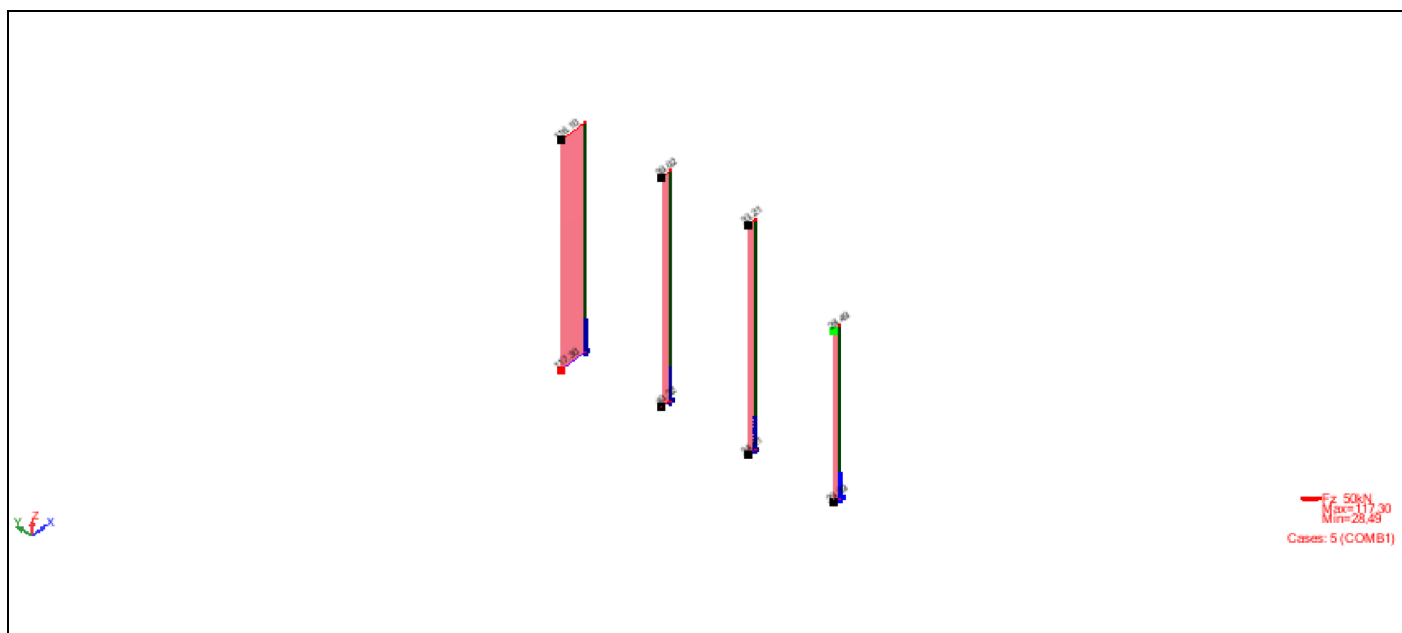
Combinations

Combinations	Name	Analysis type	Combination type	Case nature	Definition
5 (C)	COMB1	Linear Combination	ULS	Structural	$(1+2)*1.35+3*1.50$
6 (C)	COMB2	Linear Combination	SLS	Structural	$(1+2+3)*1.00$
7	Nonlinear	Combination NL PD	ULS	Structural	$5*1.00$
8	Nonlinear	Combination NL PD	SLS	Structural	$6*1.00$

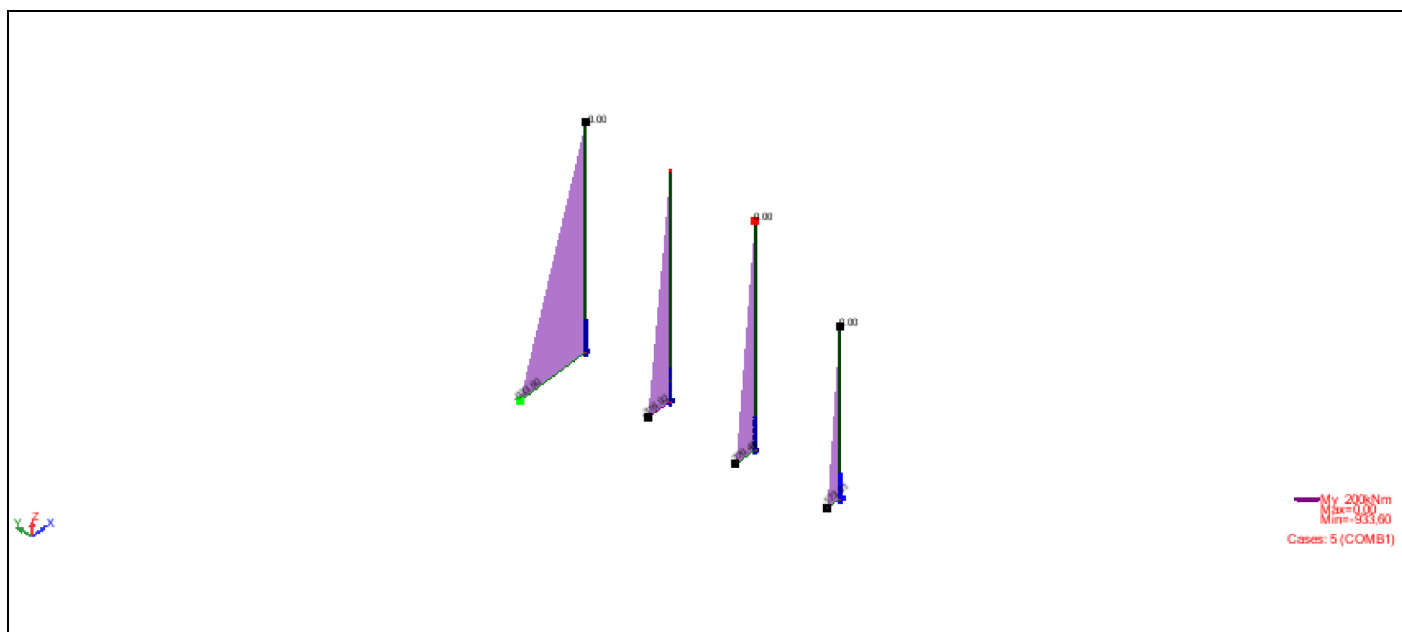
View - FX; Cases: 5 (COMB1)



View - FZ; Cases: 5 (COMB1)



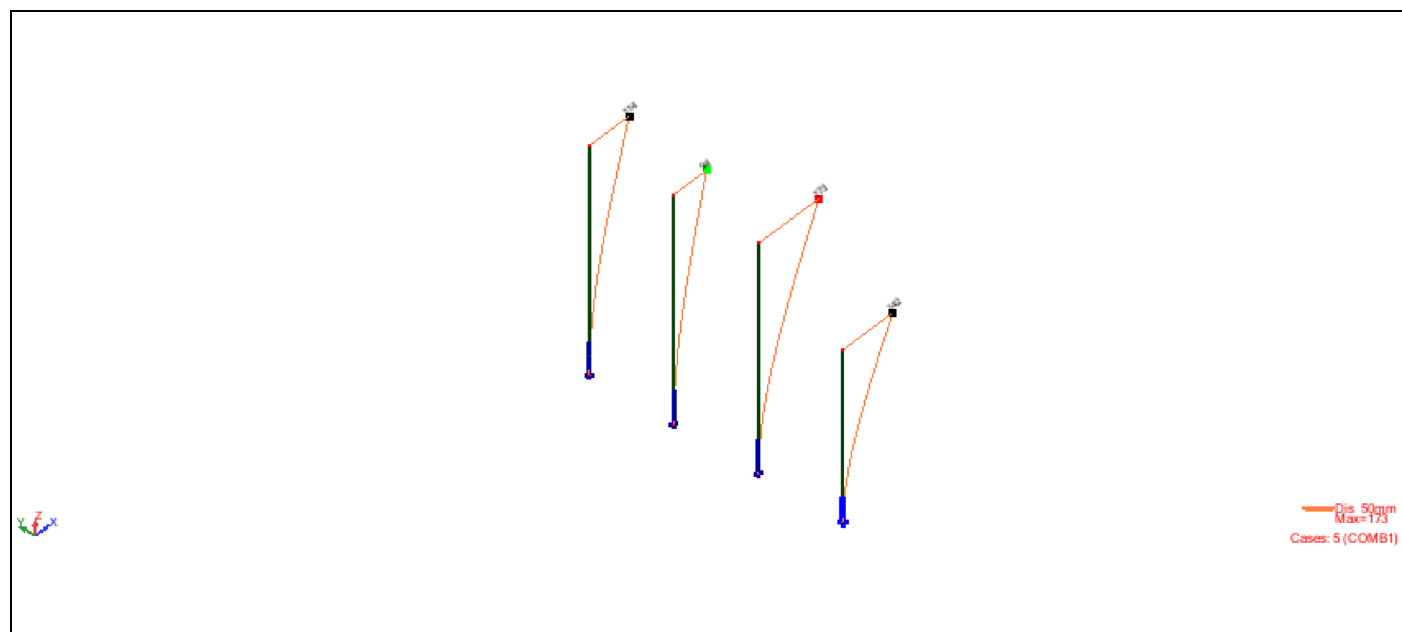
View - MY; Cases: 5 (COMB1)



Member Forces ULS: envelope

Bar	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
2 / MAX	15,56	0,0	40,33	0,0	0,00	0,0
Node	5	5	5	5	6	5
Case	5 (C)	1	7	1	5 (C)	1
2 / MIN	0,0	0,0	0,0	0,0	-317,81	0,0
Node	5	5	5	5	5	5
Case	3	1	1	1	7	1
3 / MAX	27,25	0,0	117,57	0,0	0,00	0,0
Node	7	7	7	7	8	7
Case	5 (C)	1	7	1	3	1
3 / MIN	0,0	0,0	0,0	0,0	-935,73	0,0
Node	7	7	7	7	7	7
Case	3	1	1	1	7	1
8 / MAX	12,39	0,0	34,57	0,0	0,00	0,0
Node	3	3	3	3	15	3
Case	5 (C)	1	7	1	5 (C)	1
8 / MIN	0,0	0,0	-0,00	0,0	-271,78	0,0
Node	3	3	15	3	3	3
Case	3	1	3	1	7	1
10 / MAX	26,08	0,0	29,94	0,0	0,00	0,0
Node	11	11	11	11	14	11
Case	5 (C)	1	7	1	5 (C)	1
10 / MIN	0,0	0,0	0,0	0,0	-176,95	0,0
Node	11	11	11	11	11	11
Case	3	1	1	1	7	1

View - Exact deformation(s); Cases: 5 (COMB1)



Displacements SLS: global extremes

	UX (mm)	UY (mm)	UZ (mm)	RX (Rad)	RY (Rad)	RZ (Rad)
MAX	128	0,0	0,0	0,0	0,027	0,0
Node	15	3	3	3	14	3
Case	8	1	1	1	8	1
MIN	0,0	0,0	-1	0,0	0,0	0,0
Node	3	3	15	3	3	3
Case	1	1	8	1	1	1

Reactions ULS: global extremes

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	0,0	0,0	27,25	0,0	0,0	0,0
Node	3	3	7	3	3	3
Case	1	1	5 (C)	1	1	1
MIN	-117,30	0,0	0,0	0,0	-935,73	0,0
Node	7	3	3	3	7	3
Case	7	1	3	1	7	1

Stresses - Global extremes

	S max (MPa)	S min (MPa)	S max(My) (MPa)
MAX	340,69	2,53	337,62
Bar	10	10	10
Node	11	14	11
Case	7	5 (C)	7
MIN	0,0	-335,01	0,0
Bar	2	3	2
Node	6	7	5
Case	1	7	1

	S max(Mz) (MPa)	S min(My) (MPa)	S min(Mz) (MPa)	Fx/Ax (MPa)
MAX	0,0	0,0	0,0	3,16
Bar	2	2	2	10
Node	5	5	5	11
Case	1	1	1	5 (C)
MIN	0,0	-337,62	0,0	0,0
Bar	2	10	2	2
Node	5	11	5	5
Case	1	7	1	3

Member Verification

STEEL DESIGN

CODE: EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.

ANALYSIS TYPE: Member Verification

CODE GROUP:

MEMBER: 2 Column_1

POINT: 1

COORDINATE: x = 0.00 L = 0.00 m

LOADS:

Governing Load Case: 7 Nonlinear 5*1.00

MATERIAL:

S355 (S355) $f_y = 355.00$ MPa



SECTION PARAMETERS: KH 457x10

h=457 mm

gM0=1.00

gM1=1.00

Ay=8913 mm²

Az=8913 mm²

Ax=14000 mm²

tw=10 mm

Iy=350910000 mm⁴

Iz=350910000 mm⁴

Ix=701830000 mm⁴

Wply=1998423 mm³

Wplz=1998423 mm³

INTERNAL FORCES AND CAPACITIES:

N_{Ed} = 15.09 kN

M_{y,Ed} = -317.81 kN*m

N_{c,Rd} = 4970.00 kN

M_{y,Ed,max} = -317.81 kN*m

N_{b,Rd} = 2232.53 kN

M_{y,c,Rd} = 709.44 kN*m

MN_{y,Rd} = 709.40 kN*m

V_{z,Ed} = 40.33 kN

V_{z,c,Rd} = 1826.74 kN

Class of section = 2



LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:



About y axis:

L_y = 8.00 m

Lam_y = 1.34

L_{cr,y} = 16.00 m

X_y = 0.45

Lam_y = 101.06

k_{yy} = 0.79



About z axis:

L_z = 8.00 m

Lam_z = 1.34

L_{cr,z} = 16.00 m

X_z = 0.45

Lam_z = 101.06

k_{zy} = 0.48

VERIFICATION FORMULAS:

Section strength check:

N_{Ed}/N_{c,Rd} = 0.00 < 1.00 (6.2.4.(1))

M_{y,Ed}/M_{y,c,Rd} = 0.45 < 1.00 (6.2.5.(1))

M_{y,Ed}/MN_{y,Rd} = 0.45 < 1.00 (6.2.9.1.(2))

V_{z,Ed}/V_{z,c,Rd} = 0.02 < 1.00 (6.2.6.(1))

Global stability check of member:

Lam_{b,y} = 101.06 < Lam_{b,max} = 210.00 Lam_{b,z} = 101.06 < Lam_{b,max} = 210.00 STABLE

N_{Ed}/(X_y*N_{Rk}/gM1) + k_{yy}*M_{y,Ed,max}/(XLT*M_{y,Rk}/gM1) = 0.36 < 1.00 (6.3.3.(4))

N_{Ed}/(X_z*N_{Rk}/gM1) + k_{zy}*M_{y,Ed,max}/(XLT*M_{y,Rk}/gM1) = 0.22 < 1.00 (6.3.3.(4))

STEEL DESIGN

CODE: EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.

ANALYSIS TYPE: Member Verification

CODE GROUP:

MEMBER: 3 Column_1

POINT: 1

COORDINATE: x = 0.00 L = 0.00 m

LOADS:

Governing Load Case: 7 Nonlinear 5*1.00

MATERIAL:

S355 (S355) $f_y = 355.00$ MPa



SECTION PARAMETERS: KH 610x10

h=610 mm

$gM_0=1.00$

$gM_1=1.00$

$A_y=11968$ mm²

$A_z=11968$ mm²

$A_x=18800$ mm²

tw=10 mm

$I_y=848470000$ mm⁴

$I_z=848470000$ mm⁴

$I_x=1696930000$ mm⁴

$W_{ely}=2781869$ mm³

$W_{elz}=2781869$ mm³

$A_{eff}=18800$ mm²

$W_{eff,y}=2781869$ mm³

Attention: Section of the 4 class! The program does not perform a full analysis of the 4 class for these section types; they are treated as the 3 class sections.

INTERNAL FORCES AND CAPACITIES:

$N_{Ed} = 25.57$ kN

$M_{y,Ed} = -935.73$ kN*m

$N_{c,Rd} = 6674.00$ kN

$M_{y,Ed,max} = -935.73$ kN*m

$N_{b,Rd} = 4453.30$ kN

$M_{y,c,Rd} = 987.56$ kN*m

$V_{z,Ed} = 117.57$ kN

$V_{z,c,Rd} = 2453.05$ kN

Class of section = 3



LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:



About y axis:

$L_y = 8.00$ m

$\lambda_{m,y} = 1.00$

$L_{cr,y} = 16.00$ m

$\chi_y = 0.67$

$\lambda_{my} = 75.31$

$\chi_{yy} = 0.79$



About z axis:

$L_z = 8.00$ m

$\lambda_{m,z} = 1.00$

$L_{cr,z} = 16.00$ m

$\chi_z = 0.67$

$\lambda_{mz} = 75.31$

$\chi_{zy} = 0.79$

VERIFICATION FORMULAS:

Section strength check:

$M_{y,Ed}/M_{y,c,Rd} = 0.95 < 1.00$ (6.2.5.(1))

$N_{Ed}/N_{c,Rd} + M_{y,Ed}/M_{y,c,Rd} = 0.95 < 1.00$ (6.2.1(7))

$V_{z,Ed}/V_{z,c,Rd} = 0.05 < 1.00$ (6.2.6.(1))

Global stability check of member:

$\lambda_{m,y} = 75.31 < \lambda_{m,max} = 210.00$ $\lambda_{m,z} = 75.31 < \lambda_{m,max} = 210.00$ STABLE

$N_{Ed}/(\chi_y \cdot N_{Rk}/gM_1) + \chi_{yy} \cdot M_{y,Ed,max}/(XLT \cdot M_{Rk}/gM_1) = 0.76 < 1.00$ (6.3.3.(4))

$N_{Ed}/(\chi_z \cdot N_{Rk}/gM_1) + \chi_{zy} \cdot M_{y,Ed,max}/(XLT \cdot M_{Rk}/gM_1) = 0.76 < 1.00$ (6.3.3.(4))

STEEL DESIGN

CODE: EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.

ANALYSIS TYPE: Member Verification

CODE GROUP:

MEMBER: 8

POINT: 1

COORDINATE: x = 0.00 L = 0.00 m

LOADS:

Governing Load Case: 7 Nonlinear 5*1.00

MATERIAL:

S355 (S355) $f_y = 355.00$ MPa



SECTION PARAMETERS: KH 355.6x10

h=356 mm

$g_{M0}=1.00$

$g_{M1}=1.00$

$A_y=6939$ mm²

$A_z=6939$ mm²

$A_x=10900$ mm²

tw=10 mm

$I_y=162230000$ mm⁴

$I_z=162230000$ mm⁴

$I_x=324470000$ mm⁴

$W_{ply}=1194727$ mm³

$W_{plz}=1194727$ mm³

INTERNAL FORCES AND CAPACITIES:

$N_{Ed} = 11.65$ kN

$M_{y,Ed} = -271.78$ kN*m

$N_{c,Rd} = 3869.50$ kN

$M_{y,Ed,max} = -271.78$ kN*m

$N_{b,Rd} = 1114.69$ kN

$M_{y,c,Rd} = 424.13$ kN*m

$M_{N,y,Rd} = 424.11$ kN*m

$V_{z,Ed} = 34.57$ kN

$V_{z,c,Rd} = 1422.24$ kN

Class of section = 2



LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:



About y axis:

$L_y = 8.00$ m

$\lambda_{m,y} = 1.74$

$L_{cr,y} = 16.00$ m

$\chi_y = 0.29$

$\lambda_{my} = 131.15$

$\eta_{yy} = 0.79$



About z axis:

$L_z = 8.00$ m

$\lambda_{m,z} = 1.74$

$L_{cr,z} = 16.00$ m

$\chi_z = 0.29$

$\lambda_{mz} = 131.15$

$\eta_{zy} = 0.48$

VERIFICATION FORMULAS:

Section strength check:

$N_{Ed}/N_{c,Rd} = 0.00 < 1.00$ (6.2.4.(1))

$M_{y,Ed}/M_{y,c,Rd} = 0.64 < 1.00$ (6.2.5.(1))

$M_{y,Ed}/M_{N,y,Rd} = 0.64 < 1.00$ (6.2.9.1.(2))

$V_{z,Ed}/V_{z,c,Rd} = 0.02 < 1.00$ (6.2.6.(1))

Global stability check of member:

$\lambda_{m,y} = 131.15 < \lambda_{m,max} = 210.00$ $\lambda_{m,z} = 131.15 < \lambda_{m,max} = 210.00$ STABLE

$N_{Ed}/(\chi_y N_{Rk}/g_{M1}) + \eta_{yy} M_{y,Ed,max}/(XLT M_{y,Rk}/g_{M1}) = 0.52 < 1.00$ (6.3.3.(4))

$N_{Ed}/(\chi_z N_{Rk}/g_{M1}) + \eta_{zy} M_{y,Ed,max}/(XLT M_{y,Rk}/g_{M1}) = 0.32 < 1.00$ (6.3.3.(4))

STEEL DESIGN

CODE: EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.

ANALYSIS TYPE: Member Verification

CODE GROUP:

MEMBER: 10

POINT: 1

COORDINATE: x = 0.00 L = 0.00 m

LOADS:

Governing Load Case: 7 Nonlinear 5*1.00

MATERIAL:

S355 (S355) $f_y = 355.00$ MPa



SECTION PARAMETERS: KH 273x10

h=273 mm

$gM_0=1.00$

$gM_1=1.00$

$A_y=5258$ mm²

$A_z=5258$ mm²

$A_x=8260$ mm²

tw=10 mm

$I_y=71540000$ mm⁴

$I_z=71540000$ mm⁴

$I_x=143080000$ mm⁴

$W_{ply}=692023$ mm³

$W_{plz}=692023$ mm³

INTERNAL FORCES AND CAPACITIES:

$N_{Ed} = 25.37$ kN

$M_{y,Ed} = -176.95$ kN*m

$N_{c,Rd} = 2932.30$ kN

$M_{y,Ed,max} = -176.95$ kN*m

$N_{b,Rd} = 870.73$ kN

$M_{y,c,Rd} = 245.67$ kN*m

$V_{z,Ed} = 29.94$ kN

$M_{N,y,Rd} = 245.59$ kN*m

$V_{z,c,Rd} = 1077.77$ kN

Class of section = 1



LATERAL BUCKLING PARAMETERS:

BUCKLING PARAMETERS:



About y axis:

$L_y = 6.00$ m

$\lambda_{m,y} = 1.71$

$L_{cr,y} = 12.00$ m

$X_y = 0.30$

$\lambda_{my} = 128.94$

$k_{yy} = 0.80$



About z axis:

$L_z = 6.00$ m

$\lambda_{m,z} = 1.71$

$L_{cr,z} = 12.00$ m

$X_z = 0.30$

$\lambda_{mz} = 128.94$

$k_{zy} = 0.48$

VERIFICATION FORMULAS:

Section strength check:

$N_{Ed}/N_{c,Rd} = 0.01 < 1.00$ (6.2.4.(1))

$M_{y,Ed}/M_{y,c,Rd} = 0.72 < 1.00$ (6.2.5.(1))

$M_{y,Ed}/M_{N,y,Rd} = 0.72 < 1.00$ (6.2.9.1.(2))

$V_{z,Ed}/V_{z,c,Rd} = 0.03 < 1.00$ (6.2.6.(1))

Global stability check of member:

$\lambda_{m,y} = 128.94 < \lambda_{m,max} = 210.00$ $\lambda_{m,z} = 128.94 < \lambda_{m,max} = 210.00$ STABLE

$N_{Ed}/(X_y \cdot N_{Rk}/gM_1) + k_{yy} \cdot M_{y,Ed,max}/(XLT \cdot M_{y,Rk}/gM_1) = 0.60 < 1.00$ (6.3.3.(4))

$N_{Ed}/(X_z \cdot N_{Rk}/gM_1) + k_{zy} \cdot M_{y,Ed,max}/(XLT \cdot M_{y,Rk}/gM_1) = 0.38 < 1.00$ (6.3.3.(4))



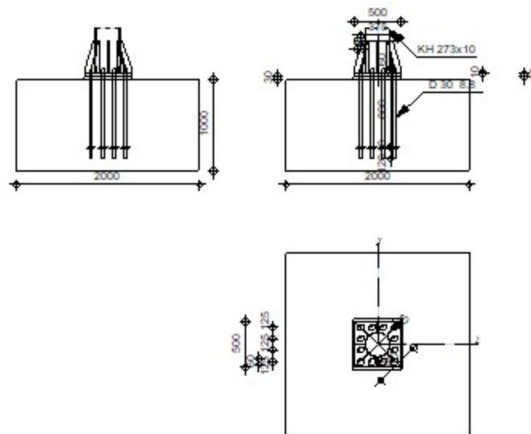
Autodesk Robot Structural Analysis Professional 2019

Fixed column base design

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



Ratio
 0,91



GENERAL

Connection no.: 5
 Connection name: Fixed column base
 Structure node: 11
 Structure bars: 10

GEOMETRY

COLUMN

Section: KH 273x10

Bar no.: 10

$L_c =$	6,00	[m]	Column length
$\alpha =$	0,0	[Deg]	Inclination angle
$h_c =$	273	[mm]	Height of column section
$b_{fc} =$	273	[mm]	Width of column section
$t_{wc} =$	10	[mm]	Thickness of the web of column section
$t_{fc} =$	10	[mm]	Thickness of the flange of column section
$r_c =$	0	[mm]	Radius of column section fillet
$A_c =$	8260	[mm ²]	Cross-sectional area of a column
$I_{yc} =$	71540000	[mm ⁴]	Moment of inertia of the column section
Material: S355			
$f_{yc} =$	355,00	[MPa]	Resistance
$f_{uc} =$	490,00	[MPa]	Yield strength of a material

COLUMN BASE

$l_{pd} =$	500	[mm]	Length
$b_{pd} =$	500	[mm]	Width
$t_{pd} =$	30	[mm]	Thickness

Material: S 355

$f_{ypd} = 355,00$ [MPa] Resistance

$f_{upd} = 470,00$ [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

$d = 30$ [mm] Bolt diameter

$A_s = 561$ [mm²] Effective section area of a bolt

$A_v = 707$ [mm²] Area of bolt section

$n_H = 4$ Number of bolt columns

$n_V = 4$ Number of bolt rows

Horizontal spacing $e_{Hi} = 125; 125$ [mm]

Vertical spacing $e_{Vi} = 125; 125$ [mm]

Anchor dimensions

$L_1 = 60$ [mm]

$L_2 = 800$ [mm]

$L_3 = 120$ [mm]

Anchor plate

$l_p = 100$ [mm] Length

$b_p = 100$ [mm] Width

$t_p = 10$ [mm] Thickness

Material: S 355

$f_y = 355,00$ [MPa] Resistance

Washer

$l_{wd} = 60$ [mm] Length

$b_{wd} = 60$ [mm] Width

$t_{wd} = 10$ [mm] Thickness

STIFFENER

$l_s = 500$ [mm] Length

$h_s = 350$ [mm] Height

$t_s = 10$ [mm] Thickness

$d_1 = 25$ [mm] Cut

$d_2 = 50$ [mm] Cut

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 = 2000 [mm] Spread footing length
 B = 2000 [mm] Spread footing width
 H = 1000 [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

$a_p = 8$ [mm] Footing plate of the column base

$a_s = 6$ [mm] Stiffeners

LOADS

Case: 7: Nonlinear 5*1.00

$N_{j,Ed} = -26,08$ [kN] Axial force

$V_{j,Ed,z} = -28,49$ [kN] Shear force

$M_{j,Ed,y} = 174,21$ [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)]

$f_j = 33,33$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

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

$c = 57$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

$b_{eff} = 123$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 386$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$A_{c0} = 47503$ [mm²] Area of the joint between the base plate and the foundation EN 1992-1:[6.7.(3)]

$A_{c1} = 427525$ [mm²] Maximum design area of load distribution EN 1992-1:[6.7.(3)]

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

$F_{rd} = 2375,14$ [kN] Bearing resistance of concrete EN 1992-1:[6.7.(3)]

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

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

$f_{jd} = 33,33$ [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n} = 185575$ [mm²] Bearing area for compression [6.2.8.2.(1)]

$A_{c,y} = 85777$ [mm²] Bearing area for bending My [6.2.8.3.(1)]

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

$F_{c,Rd,n} = 6185,83$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]
 $F_{c,Rd,y} = 2859,22$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL = 1,00$ Section class EN 1993-1-1:[5.5.2]
 $W_{pl,y} = 2120451$ [mm³] Plastic section modulus EN1993-1-1:[6.2.5.(2)]
 $M_{c,Rd,y} = 752,76$ [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]
 $h_{f,y} = 258$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]
 $F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$
 $F_{c,fc,Rd,y} = 2921,07$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$N_{j,Rd} = F_{c,Rd,n}$
 $N_{j,Rd} = 6185,83$ [kN] Resistance of a spread footing for axial compression [6.2.8.2.(1)]
 $F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$
 $F_{C,Rd,y} = 2859,22$ [kN] Resistance of spread footing in the compression zone [6.2.8.3]

TENSION ZONE

STEEL FAILURE

$A_b = 561$ [mm²] Effective anchor area [Table 3.4]
 $f_{ub} = 800,00$ [MPa] Tensile strength of the anchor material [Table 3.4]
 $\beta = 0,85$ Reduction factor of anchor resistance [3.6.1.(3)]
 $F_{t,Rd,s1} = \beta * 0.9 * f_{ub} * A_b / \gamma_{M2}$
 $F_{t,Rd,s1} = 274,67$ [kN] Anchor resistance to steel failure [Table 3.4]
 $\gamma_{Ms} = 1,20$ Partial safety factor CEB [3.2.3.2]
 $f_{yb} = 640,00$ [MPa] Yield strength of the anchor material CEB [9.2.2]
 $F_{t,Rd,s2} = f_{yb} * A_b / \gamma_{Ms}$
 $F_{t,Rd,s2} = 299,20$ [kN] Anchor resistance to steel failure CEB [9.2.2]
 $F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$
 $F_{t,Rd,s} = 274,67$ [kN] Anchor resistance to steel failure

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = F_{t,Rd,s}$
 $F_{t,Rd} = 274,67$ [kN] Tensile resistance of an anchor

BENDING OF THE BASE PLATE

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

$l_{eff,1} = 250$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]
 $l_{eff,2} = 250$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]
 $m = 120$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]
 $M_{pl,1,Rd} = 19,97$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]
 $M_{pl,2,Rd} = 19,97$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]
 $F_{T,1,Rd} = 667,77$ [kN] Resistance of a plate for mode 1 [6.2.4]
 $F_{T,2,Rd} = 596,35$ [kN] Resistance of a plate for mode 2 [6.2.4]
 $F_{T,3,Rd} = 1098,66$ [kN] Resistance of a plate for mode 3 [6.2.4]

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

$$F_{t,pl,Rd,y} = 596,35 \quad [\text{kN}] \quad \text{Tension resistance of a plate} \quad [6.2.4]$$

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

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

$$F_{T,Rd,y} = 596,35 \quad [\text{kN}] \quad \text{Resistance of a column base in the tension zone} \quad [6.2.8.3]$$

CONNECTION CAPACITY CHECK

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

$$e_y = 6681 \quad [\text{mm}] \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 129 \quad [\text{mm}] \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 188 \quad [\text{mm}] \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 192,37 \quad [\text{kN}\cdot\text{m}] \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \quad (6.23) \quad 0,91 < 1,00 \quad \text{verified} \quad (0,91)$$

SHEAR

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

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

$$\alpha_{d,z} = 0,65 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0,65 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 2,50 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

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

$$F_{1,vb,Rd,z} = 550,78 \quad [\text{kN}] \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

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

$$A_{vb} = 707 \quad [\text{mm}^2] \quad \text{Area of bolt section} \quad [6.2.2.(7)]$$

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

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

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

$$F_{2,vb,Rd} = 112,19 \quad [\text{kN}] \quad \text{Shear resistance of a bolt - without lever arm} \quad [6.2.2.(7)]$$

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

$$M_{RK,s} = 1,40 \quad [\text{kN}\cdot\text{m}] \quad \text{Characteristic bending resistance of an anchor} \quad \text{CEB [9.3.2.2]}$$

$$l_{sm} = 60 \quad [\text{mm}] \quad \text{Lever arm length} \quad \text{CEB [9.3.2.2]}$$

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

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

$$F_{v,Rd,sm} = 38,79 \quad [\text{kN}] \quad \text{Shear resistance of a bolt - with lever arm} \quad \text{CEB [9.3.1]}$$

CONCRETE PRY-OUT FAILURE

$N_{Rk,c} =$	244,37	[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]
$F_{v,Rd,cp} = k_3 * N_{Rk,c} / \gamma_{Mc}$				
$F_{v,Rd,cp} =$	226,27	[kN]	Concrete resistance for pry-out failure	CEB [9.3.1]

CONCRETE EDGE FAILURE

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

$V_{Rk,c,z}^0 =$	2519,04	[kN]	Characteristic resistance of an anchor	
$\psi_{A,V,z} =$	0,55		Factor related to anchor spacing and edge distance	
$\psi_{h,V,z} =$	1,07		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 anchors in	
$\psi_{\alpha,V,z} =$	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	
$F_{v,Rd,c,z} = V_{Rk,c,z}^0 * \psi_{A,V,z} * \psi_{h,V,z} * \psi_{s,V,z} * \psi_{ec,V,z} * \psi_{\alpha,V,z} * \psi_{ucr,V,z} / \gamma_{Mc}$				
$F_{v,Rd,c,z} =$	613,28	[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)]
$N_{c,Ed} =$	26,08	[kN]	Compressive force	[6.2.2.(6)]
$F_{f,Rd} = C_{f,d} * N_{c,Ed}$				
$F_{f,Rd} =$	7,82	[kN]	Slip resistance	[6.2.2.(6)]

SHEAR CHECK

$V_{j,Rd,z} = \eta_b * \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$				
$V_{j,Rd,z} =$	473,28	[kN]	Connection resistance for shear	CEB [9.3.1]
$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$	0,06	<	1,00	verified (0,06)

STIFFENER CHECK

Radial stiffeners

$M_1 =$	11,74	[kN*m]	Bending moment acting on a stiffener	
$Q_1 =$	138,08	[kN]	Shear force acting on a stiffener	
$z_s =$	83	[mm]	Location of the neutral axis (from the plate base)	
$I_s =$	117547731	[mm ⁴]	Moment of inertia of a stiffener	
$\sigma_d =$	5,26	[MPa]	Normal stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\sigma_g =$	29,69	[MPa]	Normal stress in upper fibers	EN 1993-1-1:[6.2.1.(5)]
$\tau =$	39,45	[MPa]	Tangent stress in a stiffener	EN 1993-1-1:[6.2.1.(5)]
$\sigma_z =$	68,54	[MPa]	Equivalent stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\max(\sigma_g, \tau / (0.58), \sigma_z) / (f_{yp} / \gamma_{M0}) \leq 1.0 (6.1)$				
	0,19	<	1,00	verified (0,19)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp} =$	2,47	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	2,47	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{y } =$	0,00	[MPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
$\tau_{z } =$	-0,85	[MPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\sigma_{\perp} / (0.9 \cdot f_u / \gamma_{M2}) \leq 1.0$ (4.1)				
	0,01	<	1,00	verified (0,01)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{y }^2 + \tau_{z }^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1.0$ (4.1)				
	0,01	<	1,00	verified (0,01)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{z }^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1.0$ (4.1)				
	0,01	<	1,00	verified (0,01)

VERTICAL WELDS OF STIFFENERS

Radial stiffeners

$\sigma_{\perp} =$	33,88	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	33,88	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{ } =$	32,88	[MPa]	Parallel tangent stress	[4.5.3.(7)]
$\sigma_z =$	88,51	[MPa]	Total equivalent stress	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(\sigma_{\perp}, \tau_{ } \cdot \sqrt{3}, \sigma_z) / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)				
	0,21	<	1,00	verified (0,21)

TRANSVERSAL WELDS OF STIFFENERS

Radial stiffeners

$\sigma_{\perp} =$	71,69	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	71,69	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{ } =$	41,91	[MPa]	Parallel tangent stress	[4.5.3.(7)]
$\sigma_z =$	160,71	[MPa]	Total equivalent stress	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(\sigma_{\perp}, \tau_{ } \cdot \sqrt{3}, \sigma_z) / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)				
	0,38	<	1,00	verified (0,38)

CONNECTION STIFFNESS

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

$b_{eff} =$	123	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
$l_{eff} =$	386	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]
$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff}) / (1.275 \cdot E)}$				
$k_{13,y} =$	26	[mm]	Stiffness coeff. of compressed concrete	[Table 6.11]
$l_{eff} =$				
	250	[mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
$m =$	120	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$				
$k_{15,y} =$	3	[mm]	Stiffness coeff. of the base plate subjected to tension	[Table 6.11]
$L_b =$				
	325	[mm]	Effective anchorage depth	[Table 6.11]
$k_{16,y} = 1.6 \cdot A_b / L_b$				
$k_{16,y} =$	3	[mm]	Stiffness coeff. of an anchor subjected to tension	[Table 6.11]

$\lambda_{0,y} = 0,85$ Column slenderness [5.2.2.5.(2)]
 $S_{j,ini,y} = 13052,51$ [kN*m] Initial rotational stiffness [Table 6.12]
 $S_{j,rig,y} = 73328,50$ [kN*m] Stiffness of a rigid connection [5.2.2.5]
 $S_{j,ini,y} < S_{j,rig,y}$ SEMI-RIGID [5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

REMARKS

Stiffeners extend outside contour of the base plate. 540 [mm] > 500 [mm]

Connection conforms to the code

Ratio 0,91



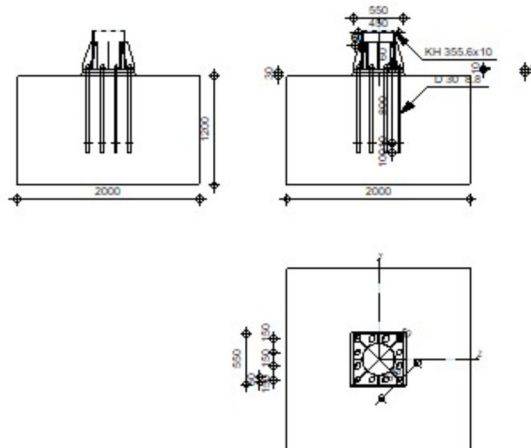
Autodesk Robot Structural Analysis Professional 2019

Fixed column base design

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



Ratio
0,97



GENERAL

Connection no.: 2
 Connection name: Fixed column base
 Structure node: 3
 Structure bars: 8

GEOMETRY

COLUMN

Section: KH 355.6x10

Bar no.: 8

$L_c = 6,00$ [m] Column length
 $\alpha = 0,0$ [Deg] Inclination angle
 $h_c = 356$ [mm] Height of column section
 $b_{fc} = 356$ [mm] Width of column section
 $t_{wc} = 10$ [mm] Thickness of the web of column section
 $t_{fc} = 10$ [mm] Thickness of the flange of column section
 $r_c = 0$ [mm] Radius of column section fillet
 $A_c = 10900$ [mm²] Cross-sectional area of a column
 $I_{yc} = 162230000$ [mm⁴] Moment of inertia of the column section
 Material: S355
 $f_{yc} = 355,00$ [MPa] Resistance
 $f_{uc} = 490,00$ [MPa] Yield strength of a material

COLUMN BASE

$l_{pd} = 550$ [mm] Length
 $b_{pd} = 550$ [mm] Width
 $t_{pd} = 40$ [mm] Thickness

Material: S 355

$f_{ypd} = 355,00$ [MPa] Resistance

$f_{upd} = 470,00$ [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

$d = 30$ [mm] Bolt diameter

$A_s = 561$ [mm²] Effective section area of a bolt

$A_v = 707$ [mm²] Area of bolt section

$n_H = 4$ Number of bolt columns

$n_V = 4$ Number of bolt rows

Horizontal spacing $e_{Hi} = 150; 150$ [mm]

Vertical spacing $e_{Vi} = 150; 150$ [mm]

Anchor dimensions

$L_1 = 60$ [mm]

$L_2 = 800$ [mm]

$L_3 = 100$ [mm]

Anchor plate

$d = 80$ [mm] Diameter

$t_p = 10$ [mm] Thickness

Material: S 355

$f_y = 355,00$ [MPa] Resistance

Washer

$l_{wd} = 60$ [mm] Length

$b_{wd} = 60$ [mm] Width

$t_{wd} = 10$ [mm] Thickness

STIFFENER

$l_s = 550$ [mm] Length

$h_s = 300$ [mm] Height

$t_s = 10$ [mm] Thickness

$d_1 = 25$ [mm] Cut

$d_2 = 50$ [mm] Cut

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 = 2000 [mm] Spread footing length
 B = 2000 [mm] Spread footing width
 H = 1200 [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

$a_p = 8$ [mm] Footing plate of the column base

$a_s = 4$ [mm] Stiffeners

LOADS

Case: 7: Nonlinear 5*1.00

$N_{j,Ed} = -12,39$ [kN] Axial force

$V_{j,Ed,z} = -33,21$ [kN] Shear force

$M_{j,Ed,y} = 266,96$ [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)]

$f_j = 33,33$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

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

$c = 75$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

$b_{eff} = 161$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 506$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$A_{c0} = 81383$ [mm²] Area of the joint between the base plate and the foundation EN 1992-1:[6.7.(3)]

$A_{c1} = 732451$ [mm²] Maximum design area of load distribution EN 1992-1:[6.7.(3)]

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

$F_{rd} = 4069,17$ [kN] Bearing resistance of concrete EN 1992-1:[6.7.(3)]

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

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

$f_{jd} = 33,33$ [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n} = 251042$ [mm²] Bearing area for compression [6.2.8.2.(1)]

$A_{c,y} = 122012$ [mm²] Bearing area for bending My [6.2.8.3.(1)]

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

$F_{c,Rd,n} = 8368,07$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]
 $F_{c,Rd,y} = 4067,05$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL = 2,00$ Section class EN 1993-1-1:[5.5.2]
 $W_{pl,y} = 2598220$ [mm³] Plastic section modulus EN1993-1-1:[6.2.5.(2)]
 $M_{c,Rd,y} = 922,37$ [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]
 $h_{f,y} = 293$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]
 $F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$
 $F_{c,fc,Rd,y} = 3146,19$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$N_{j,Rd} = F_{c,Rd,n}$
 $N_{j,Rd} = 8368,07$ [kN] Resistance of a spread footing for axial compression [6.2.8.2.(1)]
 $F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$
 $F_{C,Rd,y} = 3146,19$ [kN] Resistance of spread footing in the compression zone [6.2.8.3]

TENSION ZONE

STEEL FAILURE

$A_b = 561$ [mm²] Effective anchor area [Table 3.4]
 $f_{ub} = 800,00$ [MPa] Tensile strength of the anchor material [Table 3.4]
 $\beta = 0,85$ Reduction factor of anchor resistance [3.6.1.(3)]
 $F_{t,Rd,s1} = \beta \cdot 0.9 \cdot f_{ub} \cdot A_b / \gamma_{M2}$
 $F_{t,Rd,s1} = 274,67$ [kN] Anchor resistance to steel failure [Table 3.4]
 $\gamma_{Ms} = 1,20$ Partial safety factor CEB [3.2.3.2]
 $f_{yb} = 640,00$ [MPa] Yield strength of the anchor material CEB [9.2.2]
 $F_{t,Rd,s2} = f_{yb} \cdot A_b / \gamma_{Ms}$
 $F_{t,Rd,s2} = 299,20$ [kN] Anchor resistance to steel failure CEB [9.2.2]
 $F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$
 $F_{t,Rd,s} = 274,67$ [kN] Anchor resistance to steel failure

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = F_{t,Rd,s}$
 $F_{t,Rd} = 274,67$ [kN] Tensile resistance of an anchor

BENDING OF THE BASE PLATE

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

$l_{eff,1} = 275$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]
 $l_{eff,2} = 275$ [mm] Effective length for a single bolt for mode 2 [6.2.6.5]
 $m = 131$ [mm] Distance of a bolt from the stiffening edge [6.2.6.5]
 $M_{pl,1,Rd} = 39,05$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]
 $M_{pl,2,Rd} = 39,05$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]
 $F_{T,1,Rd} = 1189,22$ [kN] Resistance of a plate for mode 1 [6.2.4]
 $F_{T,2,Rd} = 733,58$ [kN] Resistance of a plate for mode 2 [6.2.4]
 $F_{T,3,Rd} = 1098,66$ [kN] Resistance of a plate for mode 3 [6.2.4]

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

$$F_{t,pl,Rd,y} = 733,58 \quad [\text{kN}] \quad \text{Tension resistance of a plate} \quad [6.2.4]$$

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

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

$$F_{T,Rd,y} = 733,58 \quad [\text{kN}] \quad \text{Resistance of a column base in the tension zone} \quad [6.2.8.3]$$

CONNECTION CAPACITY CHECK

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

$$e_y = 21550 \quad [\text{mm}] \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 147 \quad [\text{mm}] \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 225 \quad [\text{mm}] \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 274,46 \quad [\text{kN}\cdot\text{m}] \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \quad (6.23) \quad 0,97 < 1,00 \quad \text{verified} \quad (0,97)$$

SHEAR

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

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

$$\alpha_{d,z} = 0,52 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0,52 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 2,50 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

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

$$F_{1,vb,Rd,z} = 587,50 \quad [\text{kN}] \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

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

$$A_{vb} = 707 \quad [\text{mm}^2] \quad \text{Area of bolt section} \quad [6.2.2.(7)]$$

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

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

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

$$F_{2,vb,Rd} = 112,19 \quad [\text{kN}] \quad \text{Shear resistance of a bolt - without lever arm} \quad [6.2.2.(7)]$$

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

$$M_{Rk,s} = 1,03 \quad [\text{kN}\cdot\text{m}] \quad \text{Characteristic bending resistance of an anchor} \quad \text{CEB [9.3.2.2]}$$

$$l_{sm} = 65 \quad [\text{mm}] \quad \text{Lever arm length} \quad \text{CEB [9.3.2.2]}$$

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

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

$$F_{v,Rd,sm} = 26,35 \quad [\text{kN}] \quad \text{Shear resistance of a bolt - with lever arm} \quad \text{CEB [9.3.1]}$$

CONCRETE PRY-OUT FAILURE

$N_{Rk,c} =$	238,67	[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]
$F_{v,Rd,cp} = k_3 * N_{Rk,c} / \gamma_{Mc}$				
$F_{v,Rd,cp} =$	220,99	[kN]	Concrete resistance for pry-out failure	CEB [9.3.1]

CONCRETE EDGE FAILURE

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

$V_{Rk,c,z}^0 =$	2346,68	[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 anchors in	
$\psi_{\alpha,V,z} =$	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	
$F_{v,Rd,c,z} = V_{Rk,c,z}^0 * \psi_{A,V,z} * \psi_{h,V,z} * \psi_{s,V,z} * \psi_{ec,V,z} * \psi_{\alpha,V,z} * \psi_{ucr,V,z} / \gamma_{Mc}$				
$F_{v,Rd,c,z} =$	651,85	[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)]
$N_{c,Ed} =$	12,39	[kN]	Compressive force	[6.2.2.(6)]
$F_{f,Rd} = C_{f,d} * N_{c,Ed}$				
$F_{f,Rd} =$	3,72	[kN]	Slip resistance	[6.2.2.(6)]

SHEAR CHECK

$V_{j,Rd,z} = \eta_b * \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$				
$V_{j,Rd,z} =$	319,87	[kN]	Connection resistance for shear	CEB [9.3.1]
$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$	0,10	<	1,00	verified (0,10)

STIFFENER CHECK

Radial stiffeners

$M_1 =$	14,24	[kN*m]	Bending moment acting on a stiffener	
$Q_1 =$	165,03	[kN]	Shear force acting on a stiffener	
$z_s =$	58	[mm]	Location of the neutral axis (from the plate base)	
$I_s =$	91258060	[mm ⁴]	Moment of inertia of a stiffener	
$\sigma_d =$	2,79	[MPa]	Normal stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\sigma_g =$	44,01	[MPa]	Normal stress in upper fibers	EN 1993-1-1:[6.2.1.(5)]
$\tau =$	55,01	[MPa]	Tangent stress in a stiffener	EN 1993-1-1:[6.2.1.(5)]
$\sigma_z =$	95,32	[MPa]	Equivalent stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\max(\sigma_g, \tau / (0.58), \sigma_z) / (f_{yp} / \gamma_{M0}) \leq 1.0$ (6.1)				
	0,27	<	1,00	verified (0,27)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp} =$	4,61	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	4,61	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{y } =$	0,00	[MPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
$\tau_{z } =$	-1,23	[MPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\sigma_{\perp} / (0.9 \cdot f_u / \gamma_{M2}) \leq 1.0$ (4.1)				
	0,01	<	1,00	verified (0,01)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{y }^2 + \tau_{z }^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1.0$ (4.1)				
	0,02	<	1,00	verified (0,02)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{z }^2 + \tau_{\perp}^2)) / (f_u / (\beta_W \cdot \gamma_{M2}))} \leq 1.0$ (4.1)				
	0,02	<	1,00	verified (0,02)

VERTICAL WELDS OF STIFFENERS

Radial stiffeners

$\sigma_{\perp} =$	83,90	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	83,90	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{ } =$	68,76	[MPa]	Parallel tangent stress	[4.5.3.(7)]
$\sigma_z =$	205,78	[MPa]	Total equivalent stress	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(\sigma_{\perp}, \tau_{ } \cdot \sqrt{3}, \sigma_z) / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)				
	0,49	<	1,00	verified (0,49)

TRANSVERSAL WELDS OF STIFFENERS

Radial stiffeners

$\sigma_{\perp} =$	150,07	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	150,07	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{ } =$	89,57	[MPa]	Parallel tangent stress	[4.5.3.(7)]
$\sigma_z =$	337,86	[MPa]	Total equivalent stress	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(\sigma_{\perp}, \tau_{ } \cdot \sqrt{3}, \sigma_z) / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)				
	0,81	<	1,00	verified (0,81)

CONNECTION STIFFNESS

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

$b_{eff} =$	161	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
$l_{eff} =$	506	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]
$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff}) / (1.275 \cdot E)}$				
$k_{13,y} =$	34	[mm]	Stiffness coeff. of compressed concrete	[Table 6.11]
$l_{eff} =$				
	275	[mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
$m =$	131	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$				
$k_{15,y} =$	7	[mm]	Stiffness coeff. of the base plate subjected to tension	[Table 6.11]
$L_b =$				
	335	[mm]	Effective anchorage depth	[Table 6.11]
$k_{16,y} = 1.6 \cdot A_b / L_b$				
$k_{16,y} =$	3	[mm]	Stiffness coeff. of an anchor subjected to tension	[Table 6.11]

$\lambda_{0,y} =$	0,65	Column slenderness	[5.2.2.5.(2)]
$S_{j,ini,y} =$	18524,17 [kN*m]	Initial rotational stiffness	[Table 6.12]
$S_{j,rig,y} =$	166285,75 [kN*m]	Stiffness of a rigid connection	[5.2.2.5]
$S_{j,ini,y} < S_{j,rig,y}$	SEMI-RIGID		[5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

Connection conforms to the code

Ratio 0,97



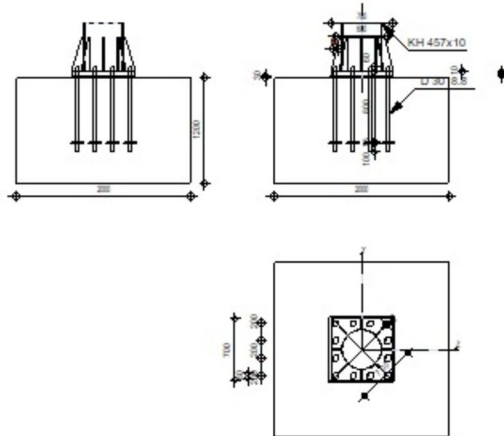
Autodesk Robot Structural Analysis Professional 2019

Fixed column base design

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



Ratio
 0,98



GENERAL

Connection no.: 3
 Connection name: Fixed column base
 Structure node: 5
 Structure bars: 2

GEOMETRY

COLUMN

Section: KH 457x10

Bar no.: 2

$L_c =$	8,00	[m]	Column length
$\alpha =$	0,0	[Deg]	Inclination angle
$h_c =$	457	[mm]	Height of column section
$b_{fc} =$	457	[mm]	Width of column section
$t_{wc} =$	10	[mm]	Thickness of the web of column section
$t_{fc} =$	10	[mm]	Thickness of the flange of column section
$r_c =$	0	[mm]	Radius of column section fillet
$A_c =$	14000	[mm ²]	Cross-sectional area of a column
$I_{yc} =$	350910000	[mm ⁴]	Moment of inertia of the column section
Material: S355			
$f_{yc} =$	355,00	[MPa]	Resistance
$f_{uc} =$	490,00	[MPa]	Yield strength of a material

COLUMN BASE

$l_{pd} =$	700	[mm]	Length
$b_{pd} =$	700	[mm]	Width
$t_{pd} =$	40	[mm]	Thickness

Material: S 355

$f_{ypd} = 355,00$ [MPa] Resistance

$f_{upd} = 470,00$ [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

$d = 30$ [mm] Bolt diameter

$A_s = 561$ [mm²] Effective section area of a bolt

$A_v = 707$ [mm²] Area of bolt section

$n_H = 4$ Number of bolt columns

$n_V = 4$ Number of bolt rows

Horizontal spacing $e_{Hi} = 200;200$ [mm]

Vertical spacing $e_{Vi} = 200;200$ [mm]

Anchor dimensions

$L_1 = 60$ [mm]

$L_2 = 800$ [mm]

$L_3 = 100$ [mm]

Anchor plate

$d = 150$ [mm] Diameter

$t_p = 15$ [mm] Thickness

Material: S 355

$f_y = 355,00$ [MPa] Resistance

Washer

$l_{wd} = 60$ [mm] Length

$b_{wd} = 60$ [mm] Width

$t_{wd} = 10$ [mm] Thickness

STIFFENER

$l_s = 700$ [mm] Length

$h_s = 400$ [mm] Height

$t_s = 10$ [mm] Thickness

$d_1 = 25$ [mm] Cut

$d_2 = 50$ [mm] Cut

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 = 2000 [mm] Spread footing length
 B = 2000 [mm] Spread footing width
 H = 1200 [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

$a_p = 8$ [mm] Footing plate of the column base

$a_s = 6$ [mm] Stiffeners

LOADS

Case: 7: Nonlinear 5*1.00

$N_{j,Ed} = -15,56$ [kN] Axial force

$V_{j,Ed,z} = -39,02$ [kN] Shear force

$M_{j,Ed,y} = 313,00$ [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)]

$f_j = 30,16$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

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

$c = 79$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

$b_{eff} = 168$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 615$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$A_{c0} = 103685$ [mm²] Area of the joint between the base plate and the foundation EN 1992-1:[6.7.(3)]

$A_{c1} = 917535$ [mm²] Maximum design area of load distribution EN 1992-1:[6.7.(3)]

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

$F_{rd} = 5140,66$ [kN] Bearing resistance of concrete EN 1992-1:[6.7.(3)]

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

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

$f_{jd} = 33,05$ [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n} = 346933$ [mm²] Bearing area for compression [6.2.8.2.(1)]

$A_{c,y} = 166346$ [mm²] Bearing area for bending My [6.2.8.3.(1)]

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

$F_{c,Rd,n} = 11467,15$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]
 $F_{c,Rd,y} = 5498,22$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

$CL = 2,00$ Section class EN 1993-1-1:[5.5.2]
 $W_{pl,y} = 4235425$ [mm³] Plastic section modulus EN1993-1-1:[6.2.5.(2)]
 $M_{c,Rd,y} = 1503,58$ [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]
 $h_{f,y} = 368$ [mm] Distance between the centroids of flanges [6.2.6.7.(1)]
 $F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$
 $F_{c,fc,Rd,y} = 4082,07$ [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$N_{j,Rd} = F_{c,Rd,n}$
 $N_{j,Rd} = 11467,15$ [kN] Resistance of a spread footing for axial compression [6.2.8.2.(1)]
 $F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$
 $F_{C,Rd,y} = 4082,07$ [kN] Resistance of spread footing in the compression zone [6.2.8.3]

TENSION ZONE

STEEL FAILURE

$A_b = 561$ [mm²] Effective anchor area [Table 3.4]
 $f_{ub} = 800,00$ [MPa] Tensile strength of the anchor material [Table 3.4]
 $\beta = 0,85$ Reduction factor of anchor resistance [3.6.1.(3)]
 $F_{t,Rd,s1} = \beta * 0.9 * f_{ub} * A_b / \gamma_{M2}$
 $F_{t,Rd,s1} = 274,67$ [kN] Anchor resistance to steel failure [Table 3.4]
 $\gamma_{Ms} = 1,20$ Partial safety factor CEB [3.2.3.2]
 $f_{yb} = 640,00$ [MPa] Yield strength of the anchor material CEB [9.2.2]
 $F_{t,Rd,s2} = f_{yb} * A_b / \gamma_{Ms}$
 $F_{t,Rd,s2} = 299,20$ [kN] Anchor resistance to steel failure CEB [9.2.2]
 $F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$
 $F_{t,Rd,s} = 274,67$ [kN] Anchor resistance to steel failure

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = F_{t,Rd,s}$
 $F_{t,Rd} = 274,67$ [kN] Tensile resistance of an anchor

BENDING OF THE BASE PLATE

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

$l_{eff,1} = 350$ [mm] Effective length for a single bolt for mode 1 [6.2.6.5]
 $l_{eff,2} = 350$ [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]
 $M_{pl,1,Rd} = 49,70$ [kN*m] Plastic resistance of a plate for mode 1 [6.2.4]
 $M_{pl,2,Rd} = 49,70$ [kN*m] Plastic resistance of a plate for mode 2 [6.2.4]
 $F_{T,1,Rd} = 1064,74$ [kN] Resistance of a plate for mode 1 [6.2.4]
 $F_{T,2,Rd} = 651,98$ [kN] Resistance of a plate for mode 2 [6.2.4]
 $F_{T,3,Rd} = 1098,66$ [kN] Resistance of a plate for mode 3 [6.2.4]

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

$$F_{t,pl,Rd,y} = 651,98 \quad [\text{kN}] \quad \text{Tension resistance of a plate} \quad [6.2.4]$$

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

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

$$F_{T,Rd,y} = 651,98 \quad [\text{kN}] \quad \text{Resistance of a column base in the tension zone} \quad [6.2.8.3]$$

CONNECTION CAPACITY CHECK

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

$$e_y = 20117 \quad [\text{mm}] \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 184 \quad [\text{mm}] \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 300 \quad [\text{mm}] \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 318,59 \quad [\text{kN}\cdot\text{m}] \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \quad (6.23) \quad 0,98 < 1,00 \quad \text{verified} \quad (0,98)$$

SHEAR

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

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

$$\alpha_{d,z} = 0,52 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0,52 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 2,50 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

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

$$F_{1,vb,Rd,z} = 587,50 \quad [\text{kN}] \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

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

$$A_{vb} = 707 \quad [\text{mm}^2] \quad \text{Area of bolt section} \quad [6.2.2.(7)]$$

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

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

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

$$F_{2,vb,Rd} = 112,19 \quad [\text{kN}] \quad \text{Shear resistance of a bolt - without lever arm} \quad [6.2.2.(7)]$$

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

$$M_{Rk,s} = 1,18 \quad [\text{kN}\cdot\text{m}] \quad \text{Characteristic bending resistance of an anchor} \quad \text{CEB [9.3.2.2]}$$

$$l_{sm} = 65 \quad [\text{mm}] \quad \text{Lever arm length} \quad \text{CEB [9.3.2.2]}$$

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

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

$$F_{v,Rd,sm} = 30,33 \quad [\text{kN}] \quad \text{Shear resistance of a bolt - with lever arm} \quad \text{CEB [9.3.1]}$$

CONCRETE PRY-OUT FAILURE

$N_{Rk,c} =$	226,83	[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]
$F_{v,Rd,cp} = k_3 * N_{Rk,c} / \gamma_{Mc}$				
$F_{v,Rd,cp} =$	210,02	[kN]	Concrete resistance for pry-out failure	CEB [9.3.1]

CONCRETE EDGE FAILURE

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

$V_{Rk,c,z}^0 =$	2014,41	[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 anchors in	
$\psi_{\alpha,V,z} =$	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	
$F_{v,Rd,c,z} = V_{Rk,c,z}^0 * \psi_{A,V,z} * \psi_{h,V,z} * \psi_{s,V,z} * \psi_{ec,V,z} * \psi_{\alpha,V,z} * \psi_{ucr,V,z} / \gamma_{Mc}$				
$F_{v,Rd,c,z} =$	559,56	[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)]
$N_{c,Ed} =$	15,56	[kN]	Compressive force	[6.2.2.(6)]
$F_{f,Rd} = C_{f,d} * N_{c,Ed}$				
$F_{f,Rd} =$	4,67	[kN]	Slip resistance	[6.2.2.(6)]

SHEAR CHECK

$V_{j,Rd,z} = \eta_b * \min(F_{1,vb,Rd,z}, F_{2,vb,Rd}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{f,Rd}$				
$V_{j,Rd,z} =$	368,58	[kN]	Connection resistance for shear	CEB [9.3.1]
$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$	0,11	<	1,00	verified (0,11)

STIFFENER CHECK

Radial stiffeners

$M_1 =$	15,27	[kN*m]	Bending moment acting on a stiffener	
$Q_1 =$	213,52	[kN]	Shear force acting on a stiffener	
$z_s =$	71	[mm]	Location of the neutral axis (from the plate base)	
$I_s =$	204253026	[mm ⁴]	Moment of inertia of a stiffener	
$\sigma_d =$	2,28	[MPa]	Normal stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\sigma_g =$	27,62	[MPa]	Normal stress in upper fibers	EN 1993-1-1:[6.2.1.(5)]
$\tau =$	53,38	[MPa]	Tangent stress in a stiffener	EN 1993-1-1:[6.2.1.(5)]
$\sigma_z =$	92,48	[MPa]	Equivalent stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\max(\sigma_g, \tau / (0.58), \sigma_z) / (f_{yp} / \gamma_{M0}) \leq 1.0 (6.1)$				
	0,26	<	1,00	verified (0,26)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp} =$	2,25	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	2,25	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{y } =$	0,00	[MPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
$\tau_{z } =$	-0,81	[MPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\sigma_{\perp} / (0.9 \cdot f_u / \gamma_{M2}) \leq 1.0$ (4.1)				
	0,01	<	1,00	verified (0,01)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{y }^2 + \tau_{z }^2))} / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)				
	0,01	<	1,00	verified (0,01)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{z }^2 + \tau_{\perp}^2))} / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)				
	0,01	<	1,00	verified (0,01)

VERTICAL WELDS OF STIFFENERS

Radial stiffeners

$\sigma_{\perp} =$	33,73	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	33,73	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{ } =$	44,48	[MPa]	Parallel tangent stress	[4.5.3.(7)]
$\sigma_z =$	102,41	[MPa]	Total equivalent stress	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(\sigma_{\perp}, \tau_{ } \cdot \sqrt{3}, \sigma_z) / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)				
	0,25	<	1,00	verified (0,25)

TRANSVERSAL WELDS OF STIFFENERS

Radial stiffeners

$\sigma_{\perp} =$	103,55	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	103,55	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{ } =$	59,05	[MPa]	Parallel tangent stress	[4.5.3.(7)]
$\sigma_z =$	230,99	[MPa]	Total equivalent stress	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(\sigma_{\perp}, \tau_{ } \cdot \sqrt{3}, \sigma_z) / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)				
	0,55	<	1,00	verified (0,55)

CONNECTION STIFFNESS

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

$b_{eff} =$	168	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
$l_{eff} =$	615	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]
$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot l_{eff})} / (1.275 \cdot E)$				
$k_{13,y} =$	38	[mm]	Stiffness coeff. of compressed concrete	[Table 6.11]
$l_{eff} =$				
	350	[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]
$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$				
$k_{15,y} =$	3	[mm]	Stiffness coeff. of the base plate subjected to tension	[Table 6.11]
$L_b =$				
	335	[mm]	Effective anchorage depth	[Table 6.11]
$k_{16,y} = 1.6 \cdot A_b / L_b$				
$k_{16,y} =$	3	[mm]	Stiffness coeff. of an anchor subjected to tension	[Table 6.11]

$\lambda_{0,y} = 0,67$ Column slenderness [5.2.2.5.(2)]
 $S_{j,ini,y} = 22944,74$ [kN*m] Initial rotational stiffness [Table 6.12]
 $S_{j,rig,y} = 269762,06$ [kN*m] Stiffness of a rigid connection [5.2.2.5]
 $S_{j,ini,y} < S_{j,rig,y}$ SEMI-RIGID [5.2.2.5.(2)]

WEAKEST COMPONENT:

BASE PLATE - BENDING

REMARKS

Stiffeners extend outside contour of the base plate. 910 [mm] $>$ 700 [mm]

Connection conforms to the code

Ratio $0,98$



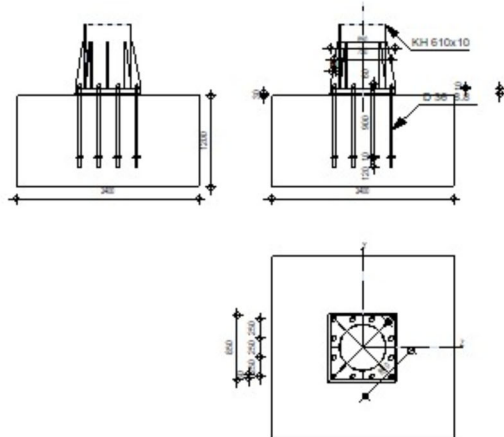
Autodesk Robot Structural Analysis Professional 2019

Fixed column base design

Eurocode 3: EN 1993-1-8:2005/AC:2009



Ratio
 0,94



GENERAL

Connection no.: 4
 Connection name: Fixed column base
 Structure node: 7
 Structure bars: 3

GEOMETRY

COLUMN

Section: KH 610x10

Bar no.: 3

$L_c = 8,00$ [m] Column length
 $\alpha = 0,0$ [Deg] Inclination angle
 $h_c = 610$ [mm] Height of column section
 $b_{fc} = 610$ [mm] Width of column section
 $t_{wc} = 10$ [mm] Thickness of the web of column section
 $t_{fc} = 10$ [mm] Thickness of the flange of column section
 $r_c = 0$ [mm] Radius of column section fillet
 $A_c = 18800$ [mm²] Cross-sectional area of a column
 $I_{yc} = 848470000$ [mm⁴] Moment of inertia of the column section

Material: S355

$f_{yc} = 355,00$ [MPa] Resistance
 $f_{uc} = 490,00$ [MPa] Yield strength of a material

COLUMN BASE

$l_{pd} = 850$ [mm] Length
 $b_{pd} = 850$ [mm] Width
 $t_{pd} = 70$ [mm] Thickness
 Material: S 355

$f_{ypd} = 335,00$ [MPa] Resistance
 $f_{upd} = 450,00$ [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
 $d = 36$ [mm] Bolt diameter
 $A_s = 817$ [mm²] Effective section area of a bolt
 $A_v = 1018$ [mm²] Area of bolt section
 $n_H = 4$ Number of bolt columns
 $n_V = 4$ Number of bolt rows
 Horizontal spacing $e_{Hi} = 250; 250$ [mm]
 Vertical spacing $e_{Vi} = 250; 250$ [mm]

Anchor dimensions

$L_1 = 60$ [mm]
 $L_2 = 900$ [mm]
 $L_3 = 120$ [mm]

Anchor plate

$d = 100$ [mm] Diameter
 $t_p = 10$ [mm] Thickness
 Material: S 355
 $f_y = 355,00$ [MPa] Resistance

Washer

$l_{wd} = 60$ [mm] Length
 $b_{wd} = 60$ [mm] Width
 $t_{wd} = 10$ [mm] Thickness

STIFFENER

$l_s = 850$ [mm] Length
 $h_s = 600$ [mm] Height
 $t_s = 12$ [mm] Thickness
 $d_1 = 25$ [mm] Cut
 $d_2 = 50$ [mm] Cut

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

$L = 2400$ [mm] Spread footing length
 $B = 2400$ [mm] Spread footing width
 $H = 1200$ [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

$a_p = 10$ [mm] Footing plate of the column base

$a_s = 8$ [mm] Stiffeners

LOADS

Case: 7: Nonlinear 5*1.00

$N_{j,Ed} = -27,25$ [kN] Axial force

$V_{j,Ed,z} = -116,10$ [kN] Shear force

$M_{j,Ed,y} = 930,92$ [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)]

$f_j = 26,80$ [MPa] Design bearing resistance under the base plate [6.2.5.(7)]

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

$c = 143$ [mm] Additional width of the bearing pressure zone [6.2.5.(4)]

$b_{eff} = 273$ [mm] Effective width of the bearing pressure zone under the flange [6.2.5.(3)]

$l_{eff} = 850$ [mm] Effective length of the bearing pressure zone under the flange [6.2.5.(3)]

$A_{c0} = 231960$ [mm²] Area of the joint between the base plate and the foundation EN 1992-1:[6.7.(3)]

$A_{c1} = 1678297$ [mm²] Maximum design area of load distribution EN 1992-1:[6.7.(3)]

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

$F_{rd,u} = 10398,95$ [kN] Bearing resistance of concrete EN 1992-1:[6.7.(3)]

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

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

$f_{jd} = 29,89$ [MPa] Design bearing resistance [6.2.5.(7)]

$A_{c,n} = 683134$ [mm²] Bearing area for compression [6.2.8.2.(1)]

$A_{c,y} = 341567$ [mm²] Bearing area for bending My [6.2.8.3.(1)]

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

$F_{c,Rd,n} = 20416,98$ [kN] Bearing resistance of concrete for compression [6.2.8.2.(1)]

$F_{c,Rd,y} = 10208,49$ [kN] Bearing resistance of concrete for bending My [6.2.8.3.(1)]

COLUMN FLANGE AND WEB IN COMPRESSION

CL =	4,00	Section class	EN 1993-1-1:[5.5.2]
$W_{el,y}$ =	4967255	[mm ³] Elastic section modulus	EN1993-1-1:[6.2.5.(2)]
$M_{c,Rd,y}$ =	1763,38	[kN*m] Design resistance of the section for bending	EN1993-1-1:[6.2.5]
$h_{f,y}$ =	475	[mm] Distance between the centroids of flanges	[6.2.6.7.(1)]
$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$			
$F_{c,fc,Rd,y}$ =	3712,29	[kN] Resistance of the compressed flange and web	[6.2.6.7.(1)]

RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$N_{j,Rd} = F_{c,Rd,n}$			
$N_{j,Rd}$ =	20416,98	[kN] Resistance of a spread footing for axial compression	[6.2.8.2.(1)]
$F_{C,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$			
$F_{C,Rd,y}$ =	3712,29	[kN] Resistance of spread footing in the compression zone	[6.2.8.3]

TENSION ZONE

STEEL FAILURE

A_b =	817	[mm ²] Effective anchor area	[Table 3.4]
f_{ub} =	800,00	[MPa] Tensile strength of the anchor material	[Table 3.4]
Beta =	0,85	Reduction factor of anchor resistance	[3.6.1.(3)]
$F_{t,Rd,s1} = \beta \cdot 0.9 \cdot f_{ub} \cdot A_b / \gamma_{M2}$			
$F_{t,Rd,s1}$ =	400,00	[kN] Anchor resistance to steel failure	[Table 3.4]
$F_{t,Rd,s} = F_{t,Rd,s1}$			
$F_{t,Rd,s}$ =	400,00	[kN] Anchor resistance to steel failure	

TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd} = F_{t,Rd,s}$			
$F_{t,Rd}$ =	400,00	[kN] Tensile resistance of an anchor	

BENDING OF THE BASE PLATE

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

$l_{eff,1}$ =	314	[mm] Effective length for a single bolt for mode 1	[6.2.6.5]
$l_{eff,2}$ =	314	[mm] Effective length for a single bolt for mode 2	[6.2.6.5]
m =	79	[mm] Distance of a bolt from the stiffening edge	[6.2.6.5]
$M_{pl,1,Rd}$ =	128,94	[kN*m] Plastic resistance of a plate for mode 1	[6.2.4]
$M_{pl,2,Rd}$ =	128,94	[kN*m] Plastic resistance of a plate for mode 2	[6.2.4]
$F_{T,1,Rd}$ =	6530,83	[kN] Resistance of a plate for mode 1	[6.2.4]
$F_{T,2,Rd}$ =	2619,77	[kN] Resistance of a plate for mode 2	[6.2.4]
$F_{T,3,Rd}$ =	1600,01	[kN] Resistance of a plate for mode 3	[6.2.4]
$F_{t,pl,Rd,y} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$			
$F_{t,pl,Rd,y}$ =	1600,01	[kN] Tension resistance of a plate	[6.2.4]

RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$F_{T,Rd,y} = F_{t,pl,Rd,y}$			
$F_{T,Rd,y}$ =	1600,01	[kN] Resistance of a column base in the tension zone	[6.2.8.3]

CONNECTION CAPACITY CHECK

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

$$e_y = 34168 \text{ [mm]} \quad \text{Axial force eccentricity} \quad [6.2.8.3]$$

$$z_{c,y} = 238 \text{ [mm]} \quad \text{Lever arm } F_{C,Rd,y} \quad [6.2.8.1.(2)]$$

$$z_{t,y} = 375 \text{ [mm]} \quad \text{Lever arm } F_{T,Rd,y} \quad [6.2.8.1.(3)]$$

$$M_{j,Rd,y} = 986,88 \text{ [kN*m]} \quad \text{Connection resistance for bending} \quad [6.2.8.3]$$

$$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0 \text{ (6.23)} \quad 0,94 < 1,00 \quad \text{verified} \quad (0,94)$$

SHEAR

BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

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

$$\alpha_{d,z} = 0,44 \quad \text{Coeff. taking account of the bolt position - in the direction of shear} \quad [\text{Table 3.4}]$$

$$\alpha_{b,z} = 0,44 \quad \text{Coeff. for resistance calculation } F_{1,vb,Rd} \quad [\text{Table 3.4}]$$

$$k_{1,z} = 1,98 \quad \text{Coeff. taking account of the bolt position - perpendicularly to the direction of shear} \quad [\text{Table 3.4}]$$

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

$$F_{1,vb,Rd,z} = 789,51 \text{ [kN]} \quad \text{Resistance of an anchor bolt for bearing pressure onto the base plate} \quad [6.2.2.(7)]$$

SHEAR OF AN ANCHOR BOLT

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

$$A_{vb} = 1018 \text{ [mm}^2] \quad \text{Area of bolt section} \quad [6.2.2.(7)]$$

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

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

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

$$F_{2,vb,Rd} = 161,56 \text{ [kN]} \quad \text{Shear resistance of a bolt - without lever arm} \quad [6.2.2.(7)]$$

SPLITTING RESISTANCE

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

$$N_{c,Ed} = 27,25 \text{ [kN]} \quad \text{Compressive force} \quad [6.2.2.(6)]$$

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

$$F_{f,Rd} = 8,17 \text{ [kN]} \quad \text{Slip resistance} \quad [6.2.2.(6)]$$

SHEAR CHECK

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

$$V_{j,Rd,z} = 1946,86 \text{ [kN]} \quad \text{Connection resistance for shear}$$

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

STIFFENER CHECK

Radial stiffeners

$M_1 =$	46,29	[kN*m]	Bending moment acting on a stiffener	
$Q_1 =$	352,17	[kN]	Shear force acting on a stiffener	
$z_s =$	98	[mm]	Location of the neutral axis (from the plate base)	
$I_s =$	886004830	[mm ⁴]	Moment of inertia of a stiffener	
$\sigma_d =$	1,44	[MPa]	Normal stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\sigma_g =$	29,91	[MPa]	Normal stress in upper fibers	EN 1993-1-1:[6.2.1.(5)]
$\tau =$	48,91	[MPa]	Tangent stress in a stiffener	EN 1993-1-1:[6.2.1.(5)]
$\sigma_z =$	84,73	[MPa]	Equivalent stress on the contact surface between stiffener and plate	EN 1993-1-1:[6.2.1.(5)]
$\max(\sigma_g, \tau / (0.58), \sigma_z) / (f_{yp} / \gamma_{M0}) \leq 1.0 \text{ (6.1)}$				
	0,25	<	1,00	verified (0,25)

WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp} =$	3,45	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	3,45	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{y } =$	0,00	[MPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
$\tau_{z } =$	-1,49	[MPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\sigma_{\perp} / (0.9 * f_u / \gamma_{M2}) \leq 1.0 \text{ (4.1)}$				
	0,01	<	1,00	verified (0,01)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{y }^2 + \tau_{z }^2)) / (f_u / (\beta_W * \gamma_{M2}))} \leq 1.0 \text{ (4.1)}$				
	0,02	<	1,00	verified (0,02)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{z }^2 + \tau_{\perp}^2)) / (f_u / (\beta_W * \gamma_{M2}))} \leq 1.0 \text{ (4.1)}$				
	0,02	<	1,00	verified (0,02)

VERTICAL WELDS OF STIFFENERS

Radial stiffeners

$\sigma_{\perp} =$	34,10	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	34,10	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{ } =$	36,68	[MPa]	Parallel tangent stress	[4.5.3.(7)]
$\sigma_z =$	93,21	[MPa]	Total equivalent stress	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(\sigma_{\perp}, \tau_{ } * \sqrt{3}, \sigma_z) / (f_u / (\beta_W * \gamma_{M2})) \leq 1.0 \text{ (4.1)}$				
	0,23	<	1,00	verified (0,23)

TRANSVERSAL WELDS OF STIFFENERS

Radial stiffeners

$\sigma_{\perp} =$	129,70	[MPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp} =$	129,70	[MPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{ } =$	48,74	[MPa]	Parallel tangent stress	[4.5.3.(7)]
$\sigma_z =$	272,79	[MPa]	Total equivalent stress	[4.5.3.(7)]
$\beta_W =$	0,90		Resistance-dependent coefficient	[4.5.3.(7)]
$\max(\sigma_{\perp}, \tau_{ } * \sqrt{3}, \sigma_z) / (f_u / (\beta_W * \gamma_{M2})) \leq 1.0 \text{ (4.1)}$				
	0,68	<	1,00	verified (0,68)

CONNECTION STIFFNESS

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

$b_{eff} =$	273	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
$l_{eff} =$	850	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]

$k_{13,y} = E_c \cdot \sqrt{(b_{eff} \cdot I_{eff})} / (1.275 \cdot E)$			
$k_{13,y} =$	57	[mm]	Stiffness coeff. of compressed concrete [Table 6.11]
$l_{eff} =$	314	[mm]	Effective length for a single bolt for mode 2 [6.2.6.5]
$m =$	79	[mm]	Distance of a bolt from the stiffening edge [6.2.6.5]
$k_{15,y} = 0.425 \cdot I_{eff} \cdot t_p^3 / (m^3)$			
$k_{15,y} =$	93	[mm]	Stiffness coeff. of the base plate subjected to tension [Table 6.11]
$L_b =$	416	[mm]	Effective anchorage depth [Table 6.11]
$k_{16,y} = 1.6 \cdot A_b / L_b$			
$k_{16,y} =$	3	[mm]	Stiffness coeff. of an anchor subjected to tension [Table 6.11]
$\lambda_{0,y} =$	0,50		Column slenderness [5.2.2.5.(2)]
$S_{j,ini,y} =$	87479,91	[kN*m]	Initial rotational stiffness [Table 6.12]
$S_{j,rig,y} =$	652261,31	[kN*m]	Stiffness of a rigid connection [5.2.2.5]
$S_{j,ini,y} < S_{j,rig,y}$			SEMI-RIGID [5.2.2.5.(2)]

WEAKEST COMPONENT:

ANCHOR BOLT - RUPTURE

Connection conforms to the code

Ratio 0,94