

# **ANEXOS**

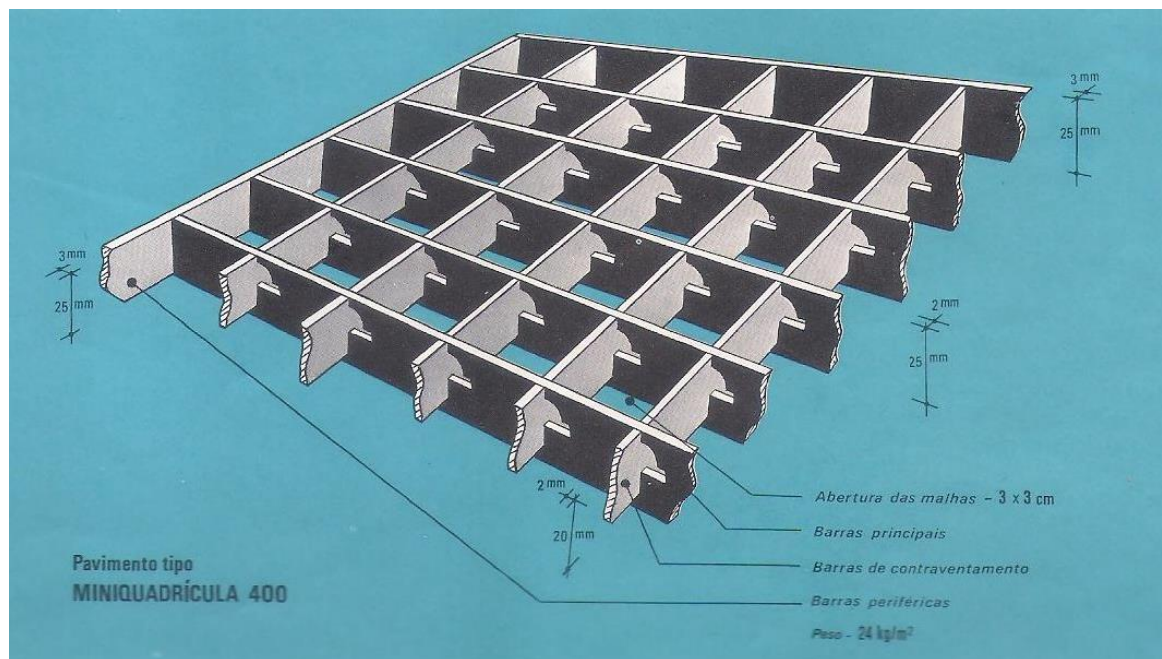


# **ANEXO I**

Pavimento Metálico Miniquadrícula 400



## Pavimento Metálico – Miniquadricula 400

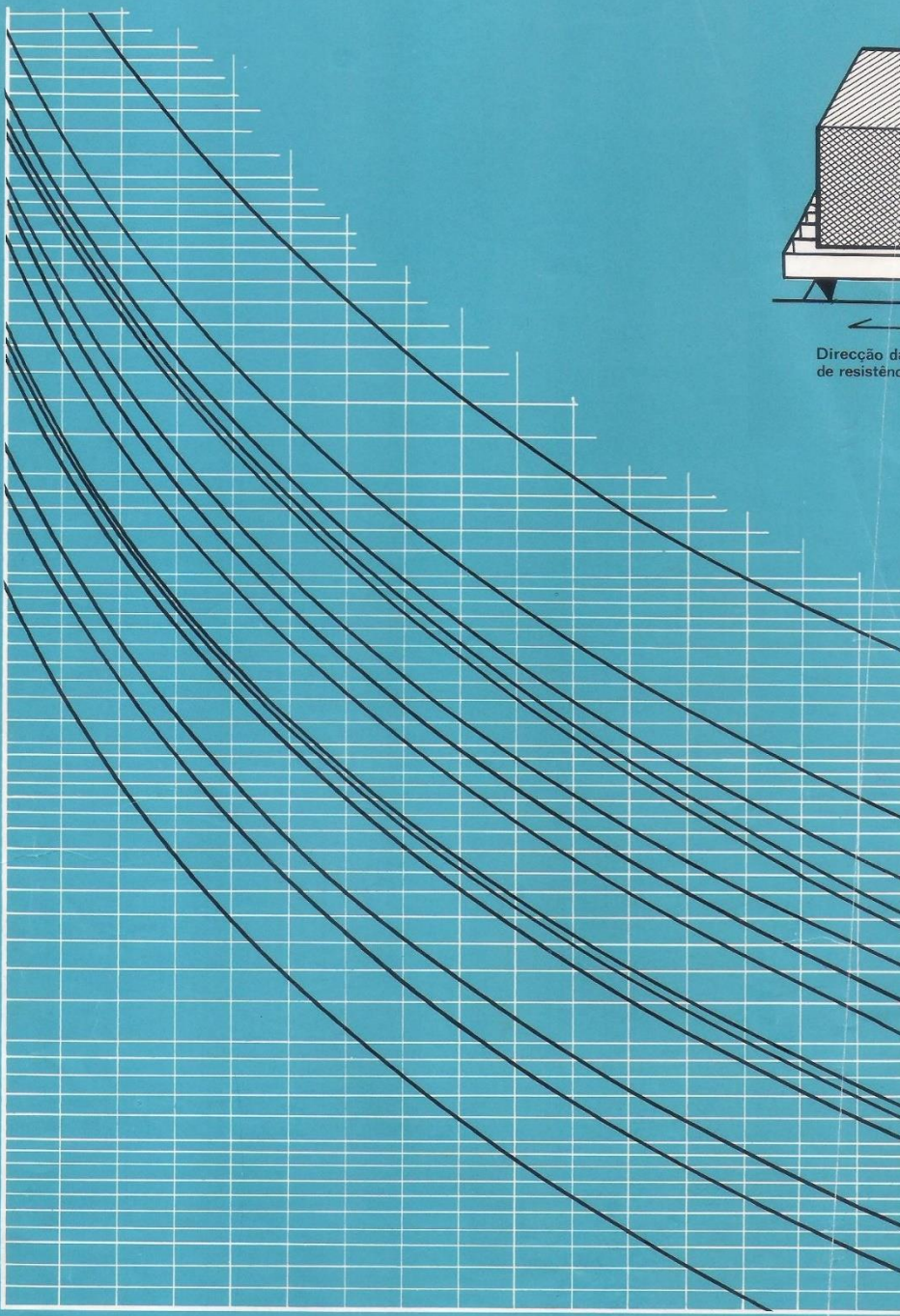


**VALORES ADMISSÍVEIS DAS SOBRECARGAS UNIFORMEMENTE DISTRIBUÍDAS (não excedendo a flecha de 1/300 do vão, nem as tensões de segurança do material)**

Sobrecargas

kg/m<sup>2</sup>

20 000  
19 000  
14 000  
12 000  
10 000  
9 000  
8 000  
7 000  
6 000  
5 000  
4 000  
3 000  
1 600  
1 400  
1 200  
1 000  
900  
800  
600  
400  
300  
200  
100



- Rectangular 6000
- Rectangular 3000
- Gradeado Losango 2300
- Rectangular 2000
- Gradeado Losango AL 2000
- Gradeado Losango 1600
- Rectangular 1400
- Gradeado Losango 1200
- Quadrícula 900
- Gradeado Losango 900
- Quadrícula 800
- Gradeado Losango 500
- Miniquadrícula 400
- Quadrícula 350



Direcção das lâminas principais de resistência.

## **ANEXO II**

Caraterísticas Geométricas e Elásticas das Chapas dos  
Revestimentos dos Alçados e da Cobertura





## FICHA TÉCNICA

CE EN 14782

### CHAPA DE AÇO TRAPEZOIDAL PERFIL 6-169-25

ESPESSURAS: 0,50 – 0,60 – 0,70 – 0,80 mm

Tipo de aço: S250 GD  
(Norma EN10346)

Tolerâncias segundo a Norma  
EN10143

Norma de fabricação: EN10169-1

POSIÇÃO A



POSIÇÃO B



#### CARACTERÍSTICAS MECÂNICAS



Reacção ao fogo: A1 (antigo M0)  
segundo a Norma EN 13501-1

ESP.	PESOS		SECÇÃO PLENA		SECÇÕES REDUZIDAS			
			C.G.	I	Face A		Face B	
	Kg/ml	Kg/m <sup>2</sup>	mm	cm <sup>4</sup>	cm <sup>4</sup>	cm <sup>3</sup>	cm <sup>4</sup>	cm <sup>3</sup>
0,5	4,91	4,84	6,55	5,56	4,23	1,92	2,36	1,52
0,6	5,89	5,81	6,55	6,68	4,64	2,31	2,93	1,90
0,7	6,87	6,77	6,55	7,79	5,60	2,80	3,52	2,30
0,8	7,85	7,74	6,55	8,90	6,61	3,33	4,14	2,73



## FICHA TÉCNICA

CE EN 14782

### CHAPA DE AÇO TRAPEZOIDAL PERFIL 6-169-25

Mapa de cargas

- o Cargas em kN/m<sup>2</sup>, considerando  $\sigma = 235$  MPa e  $f=L/200$



Vão (m)	Posição A				Posição B			
	Espessura (mm)				Espessura (mm)			
	0,5	0,6	0,7	0,8	0,5	0,6	0,7	0,8
1,50	0,962	1,061	1,270	1,502	0,515	0,652	0,773	0,912
1,75	0,588	0,650	0,775	0,917	0,307	0,393	0,462	0,546
2,00	0,378	0,420	0,497	0,589				0,340
2,25			0,329	0,391				
2,50								



Vão (m)	Posição A				Posição B			
	Espessura (mm)				Espessura (mm)			
	0,5	0,6	0,7	0,8	0,5	0,6	0,7	0,8
1,50	0,803	1,015	1,220	1,451	1,026	1,243	1,499	1,785
1,75	0,579	0,734	0,880	1,048	0,742	0,902	1,085	1,293
2,00	0,433	0,552	0,660	0,786	0,525	0,664	0,787	0,928
2,25	0,333	0,427	0,492	0,606	0,354	0,452	0,533	0,629
2,50		0,338	0,400	0,487		0,316	0,370	0,437
2,75			0,320	0,383				0,309
3,00								



Vão (m)	Posição A				Posição B			
	Espessura (mm)				Espessura (mm)			
	0,5	0,6	0,7	0,8	0,5	0,6	0,7	0,8
1,50	1,015	1,28	1,541	1,831	1,038	1,3	1,553	1,829
1,75	0,734	0,929	1,116	1,327	0,636	0,801	0,953	1,123
2,00	0,552	0,701	0,84	1	0,41	0,521	0,616	0,727
2,50	0,337	0,413	0,489	0,58				0,332
2,75		0,35	0,351	0,416				
3,00				0,303				
3,25								

## **ANEXO III**

**Relatórios de Dimensionamento das Ligações Aparafusadas**  
***"Robot Structural Analysis Professional"***





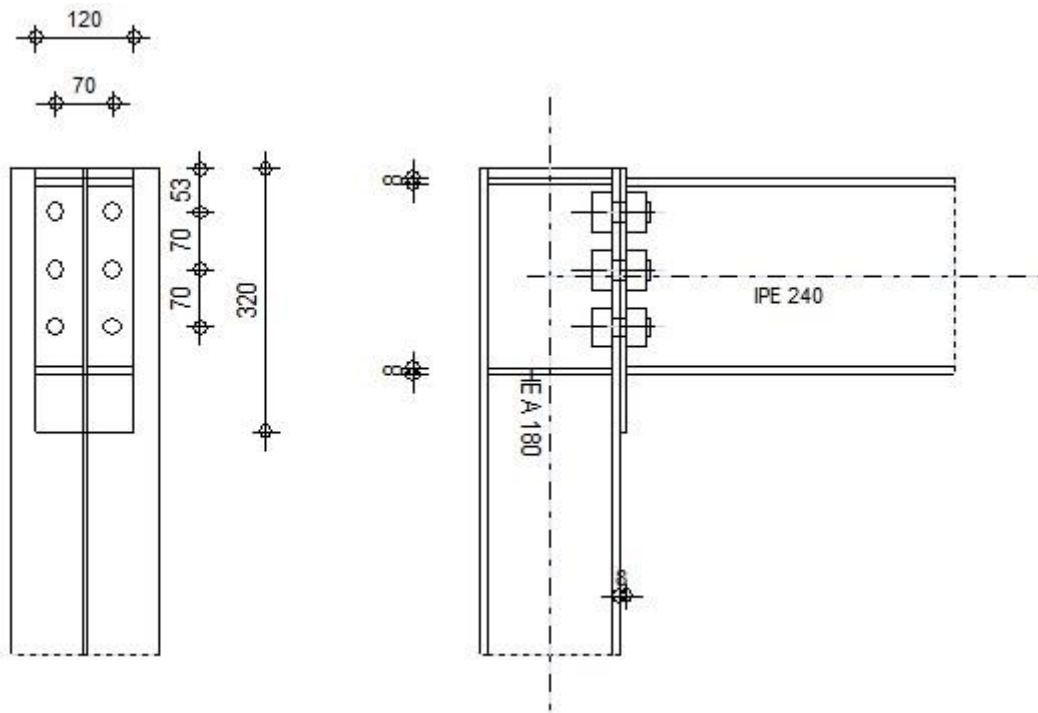
Autodesk Robot Structural Analysis Professional 2014-Student Version

## Design of fixed beam-to-column connection

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



Ratio  
0.63



### GENERAL

Connection no.: 6  
Connection name: Frame knee

### GEOMETRY

#### COLUMN

Section: HEA 180  
 $\alpha = -90.0$  [Deg] Inclination angle  
 $h_c = 171$  [mm] Height of column section  
 $b_{fc} = 180$  [mm] Width of column section  
 $t_{wc} = 6$  [mm] Thickness of the web of column section  
 $t_{fc} = 10$  [mm] Thickness of the flange of column section  
 $r_c = 15$  [mm] Radius of column section fillet  
 $A_c = 45.25$  [cm<sup>2</sup>] Cross-sectional area of a column  
 $I_{xc} = 2510.29$  [cm<sup>4</sup>] Moment of inertia of the column section  
Material: S 235  
 $f_{yc} = 235000.00$  [kPa] Resistance

## **BEAM**

Section:		IPB 240
$\alpha =$	0.0	[Deg] Inclination angle
$h_b =$	240	[mm] Height of beam section
$b_f =$	120	[mm] Width of beam section
$t_{wb} =$	6	[mm] Thickness of the web of beam section
$t_{fb} =$	10	[mm] Thickness of the flange of beam section
$r_b =$	15	[mm] Radius of beam section fillet
$r_b =$	15	[mm] Radius of beam section fillet
$A_b =$	39.12	[cm <sup>2</sup> ] Cross-sectional area of a beam
$I_{xb} =$	3891.63	[cm <sup>4</sup> ] Moment of inertia of the beam section
Material:	S 235	
$f_{yb} =$	235000.00	[kPa] Resistance

## **BOLTS**

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

$d =$	24	[mm] Bolt diameter
Class =	10.9	Bolt class
$F_{tRd} =$	254.16	[kN] Tensile resistance of a bolt
$n_h =$	2	Number of bolt columns
$n_v =$	3	Number of bolt rows
$h_1 =$	53	[mm] Distance between first bolt and upper edge of front plate
Horizontal spacing $e_i =$	70	[mm]
Vertical spacing $p_i =$	70; 70	[mm]

## **PLATE**

$h_p =$	320	[mm] Plate height
$b_p =$	120	[mm] Plate width
$t_p =$	8	[mm] Plate thickness
Material:	S 235	
$f_{yp} =$	235000.00	[kPa] Resistance

## **COLUMN STIFFENER**

### **Upper**

$h_{su} =$	152	[mm] Stiffener height
$b_{su} =$	87	[mm] Stiffener width
$t_{hu} =$	8	[mm] Stiffener thickness
Material:	S 235	
$f_{ysu} =$	235000.00	[kPa] Resistance

### **Lower**

$h_{sd} =$	152	[mm] Stiffener height
$b_{sd} =$	87	[mm] Stiffener width
$t_{hd} =$	8	[mm] Stiffener thickness
Material:	S 235	
$f_{ysu} =$	235000.00	[kPa] Resistance

## **FILLET WELDS**

$a_w =$	5	[mm] Web weld
$a_f =$	8	[mm] Flange weld
$a_s =$	5	[mm] Stiffener weld

## MATERIAL FACTORS

$\gamma_{M0} =$	1.00	Partial safety factor	[2.2]
$\gamma_{M1} =$	1.00	Partial safety factor	[2.2]
$\gamma_{M2} =$	1.25	Partial safety factor	[2.2]
$\gamma_{M3} =$	1.25	Partial safety factor	[2.2]

## LOADS

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### Ultimate limit state

Case: Manual calculations.

$M_{b1,Ed} =$	-11.80	[kN*m]	Bending moment in the right beam
$V_{b1,Ed} =$	-7.98	[kN]	Shear force in the right beam
$N_{b1,Ed} =$	-41.85	[kN]	Axial force in the right beam

## RESULTS

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### BEAM RESISTANCES

#### COMPRESSION

$A_b =$	39.12	[cm <sup>2</sup> ]	Area	EN1993-1-1:[6.2.4]
$N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$				
$N_{cb,Rd} =$	919.23	[kN]	Design compressive resistance of the section	EN1993-1-1:[6.2.4]

#### SHEAR

$A_{vb} =$	19.14	[cm <sup>2</sup> ]	Shear area	EN1993-1-1:[6.2.6.(3)]
$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3}) / \gamma_{M0}$				
$V_{cb,Rd} =$	259.74	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]
$V_{b1,Ed} / V_{cb,Rd} \leq 1,0$		0.03 < 1.00	verified	(0.03)

#### BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

$W_{plb} =$	366.68	[cm <sup>3</sup> ]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]
$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$				
$M_{b,pl,Rd} =$	86.1	[kN*m]	Plastic resistance of the section for bending (without stiffeners)	EN1993-1-1:[6.2.5.(2)]

#### BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

$W_{pl} =$	366.68	[cm <sup>3</sup> ]	Plastic section modulus	EN1993-1-1:[6.2.5]
$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$				
$M_{cb,Rd} =$	86.17	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]

#### FLANGE AND WEB - COMPRESSION

$M_{cb,Rd} =$	86.17	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
$h_f =$	230	[mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]
$F_{c,fb,Rd} = M_{cb,Rd} / h_f$				
$F_{c,fb,Rd} =$	374.32	[kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]

### COLUMN RESISTANCES

#### WEB PANEL - SHEAR

$M_{b1,Ed} =$	-11.80	[kN*m]	Bending moment (right beam)	[5.3.(3)]
$M_{b2,Ed} =$	0.00	[kN*m]	Bending moment (left beam)	[5.3.(3)]
$V_{c1,Ed} =$	0.00	[kN]	Shear force (lower column)	[5.3.(3)]

$M_{b1,Ed} = -11.80$	[kN*m]	Bending moment (right beam)	[5.3.(3)]
$V_{c2,Ed} = 0.00$	[kN]	Shear force (upper column)	[5.3.(3)]
$Z = 143$	[mm]	Lever arm	[6.2.5]
$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / Z - (V_{c1,Ed} - V_{c2,Ed}) / 2$			
$V_{wp,Ed} = -82.63$	[kN]	Shear force acting on the web panel	[5.3.(3)]

$A_{vs} = \frac{14.}{47}$	[cm <sup>2</sup> ]	Shear area of the column web	EN1993-1-1:[6.2.6.(3)]
$A_{vc} = \frac{14.}{47}$	[cm <sup>2</sup> ]	Shear area	EN1993-1-1:[6.2.6.(3)]
$d_s = 232$	[mm]	Distance between the centroids of stiffeners	[6.2.6.1.(4)]
$M_{pl,fc,Rd} = 0.9$	[kN*m]	Plastic resistance of the column flange for bending	[6.2.6.1.(4)]
$M_{pl,stu,Rd} = 0.6$	[kN*m]	Plastic resistance of the upper transverse stiffener for bending	[6.2.6.1.(4)]
$M_{pl,sti,Rd} = 0.6$	[kN*m]	Plastic resistance of the lower transverse stiffener for bending	[6.2.6.1.(4)]
$V_{wp,Rd} = 0.9 ( A_{vs} \cdot f_{y,wc} ) / (\sqrt{3} \gamma_{M0}) + \text{Min}(4 M_{pl,fc,Rd} / d_s, (2 M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,sti,Rd}) / d_s)$			
$V_{wp,Rd} = 190.77$	[kN]	Resistance of the column web panel for shear	[6.2.6.1]

$V_{wp,Ed} / V_{wp,Rd} \leq 1,0$	$0.43 < 1.00$	verified	(0.43)
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#### WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

##### Bearing:

$t_{wc} = 6$	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc} = 171$	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
$A_{vc} = 14.47$	[cm <sup>2</sup> ]	Shear area	EN1993-1-1:[6.2.6.(3)]
$\omega = 0.78$		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed} = 0.00$	[kPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
$k_{wc} = 1.00$		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]
$A_s = 13.92$	[cm <sup>2</sup> ]	Area of the web stiffener	EN1993-1-1:[6.2.4]
$F_{c,wc,Rd1} = \omega k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0} + A_s f_{ys} / \gamma_{M0}$			
$F_{c,wc,Rd1} = 514.58$	[kN]	Column web resistance	[6.2.6.2.(1)]

##### Buckling:

$d_{wc} = 122$	[mm]	Height of compressed web	[6.2.6.2.(1)]
$\lambda_p = 0.75$		Plate slenderness of an element	[6.2.6.2.(1)]
$\rho = 0.98$		Reduction factor for element buckling	[6.2.6.2.(1)]
$\lambda_s = 2.31$		Stiffener slenderness	EN1993-1-1:[6.3.1.2]
$\chi_s = 1.00$		Buckling coefficient of the stiffener	EN1993-1-1:[6.3.1.2]
$F_{c,wc,Rd2} = \omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1} + A_s \chi_s f_{ys} / \gamma_{M1}$			
$F_{c,wc,Rd2} = 510.36$	[kN]	Column web resistance	[6.2.6.2.(1)]

##### Final resistance:

$F_{c,wc,Rd,low} = \text{Min}(F_{c,wc,Rd1}, F_{c,wc,Rd2})$			
$F_{c,wc,Rd} = 510.36$	[kN]	Column web resistance	[6.2.6.2.(1)]

#### WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

##### Bearing:

$t_{wc} = 6$	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc} = 162$	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
$A_{vc} = 14.47$	[cm <sup>2</sup> ]	Shear area	EN1993-1-1:[6.2.6.(3)]
$\omega = 0.79$		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed} = 0.00$	[kPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
$k_{wc} = 1.00$		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]
$A_s = 13.92$	[cm <sup>2</sup> ]	Area of the web stiffener	EN1993-1-1:[6.2.4]

$$F_{c,wc,Rd1} = \omega k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0} + A_s f_{ys} / \gamma_{M0}$$

$$F_{c,wc,Rd1} = 508.20 \quad [\text{kN}] \quad \text{Column web resistance} \quad [6.2.6.2.(1)]$$

Buckling:

$$d_{wc} = 122 \quad [\text{mm}] \quad \text{Height of compressed web} \quad [6.2.6.2.(1)]$$

$$\lambda_p = 0.73 \quad \text{Plate slenderness of an element} \quad [6.2.6.2.(1)]$$

$$\rho = 0.99 \quad \text{Reduction factor for element buckling} \quad [6.2.6.2.(1)]$$

$$\lambda_s = 2.31 \quad \text{Stiffener slenderness} \quad \text{EN1993-1-1:[6.3.1.2]}$$

$$\chi_s = 1.00 \quad \text{Buckling coefficient of the stiffener} \quad \text{EN1993-1-1:[6.3.1.2]}$$

$$F_{c,wc,Rd2} = \omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1} + A_s \chi_s f_{ys} / \gamma_{M1}$$

$$F_{c,wc,Rd2} = 507.27 \quad [\text{kN}] \quad \text{Column web resistance} \quad [6.2.6.2.(1)]$$

Final resistance:

$$F_{c,wc,Rd,upp} = \text{Min} (F_{c,wc,Rd1}, F_{c,wc,Rd2})$$

$$F_{c,wc,Rd,upp} = 507.27 \quad [\text{kN}] \quad \text{Column web resistance} \quad [6.2.6.2.(1)]$$

## GEOMETRICAL PARAMETERS OF A CONNECTION

### EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

Nr	m	m <sub>x</sub>	e	e <sub>x</sub>	p	l <sub>eff,cp</sub>	l <sub>eff,nc</sub>	l <sub>eff,1</sub>	l <sub>eff,2</sub>	l <sub>eff,cp,g</sub>	l <sub>eff,nc,g</sub>	l <sub>eff,1,g</sub>	l <sub>eff,2,g</sub>
1	20	-	55	-	70	126	149	126	149	133	110	110	110
2	20	-	55	-	70	126	149	126	149	140	70	70	70
3	20	-	55	-	70	126	165	126	165	133	126	126	126

### EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m <sub>x</sub>	e	e <sub>x</sub>	p	l <sub>eff,cp</sub>	l <sub>eff,nc</sub>	l <sub>eff,1</sub>	l <sub>eff,2</sub>	l <sub>eff,cp,g</sub>	l <sub>eff,nc,g</sub>	l <sub>eff,1,g</sub>	l <sub>eff,2,g</sub>
1	26	-	25	-	70	165	140	140	140	152	107	107	107
2	26	-	25	-	70	165	136	136	136	140	70	70	70
3	26	-	25	-	70	165	136	136	136	152	103	103	103

m – Bolt distance from the web

m<sub>x</sub> – Bolt distance from the beam flange

e – Bolt distance from the outer edge

e<sub>x</sub> – Bolt distance from the horizontal outer edge

p – Distance between bolts

l<sub>eff,cp</sub> – Effective length for a single bolt in the circular failure mode

l<sub>eff,nc</sub> – Effective length for a single bolt in the non-circular failure mode

l<sub>eff,1</sub> – Effective length for a single bolt for mode 1

l<sub>eff,2</sub> – Effective length for a single bolt for mode 2

l<sub>eff,cp,g</sub> – Effective length for a group of bolts in the circular failure mode

l<sub>eff,nc,g</sub> – Effective length for a group of bolts in the non-circular failure mode

l<sub>eff,1,g</sub> – Effective length for a group of bolts for mode 1

l<sub>eff,2,g</sub> – Effective length for a group of bolts for mode 2

## CONNECTION RESISTANCE FOR COMPRESSION

$$N_{j,Rd} = \text{Min} ( N_{cb,Rd}, 2 F_{c,wc,Rd,low}, 2 F_{c,wc,Rd,upp} )$$

$$N_{j,Rd} = 919.23 \quad [\text{kN}] \quad \text{Connection resistance for compression} \quad [6.2]$$

$$N_{b1,Ed} / N_{j,Rd} \leq 1,0 \quad 0.05 < 1.00 \quad \text{verified} \quad (0.05)$$

## CONNECTION RESISTANCE FOR BENDING

$F_{t,Rd} = 254.16$  [kN] Bolt resistance for tension [Table 3.4]  
 $B_{p,Rd} = 156.35$  [kN] Punching shear resistance of a bolt [Table 3.4]

$F_{t,fc,Rd}$  – column flange resistance due to bending  
 $F_{t,wc,Rd}$  – column web resistance due to tension  
 $F_{t,ep,Rd}$  – resistance of the front plate due to bending  
 $F_{t,wb,Rd}$  – resistance of the web in tension

$F_{t,fc,Rd} = \text{Min} (F_{T,1,fc,Rd}, F_{T,2,fc,Rd}, F_{T,3,fc,Rd})$  [6.2.6.4] , [Tab.6.2]

$F_{t,wc,Rd} = \omega b_{eff,t,wc} t_{wc} f_{yc} / \gamma_{M0}$  [6.2.6.3.(1)]

$F_{t,ep,Rd} = \text{Min} (F_{T,1,ep,Rd}, F_{T,2,ep,Rd}, F_{T,3,ep,Rd})$  [6.2.6.5] , [Tab.6.2]

$F_{t,wb,Rd} = b_{eff,t,wb} t_{wb} f_{yb} / \gamma_{M0}$  [6.2.6.8.(1)]

### RESISTANCE OF THE BOLT ROW NO. 1

<b>F<sub>t1,Rd,comp</sub> - Formula</b>	<b>F<sub>t1,Rd,comp</sub></b>	<b>Component</b>
$F_{t1,Rd} = \text{Min} (F_{t1,Rd,comp})$	80.49	Bolt row resistance
$F_{t,fc,Rd(1)} = 133.26$	133.26	Column flange - tension
$F_{t,wc,Rd(1)} = 152.33$	152.33	Column web - tension
$F_{t,ep,Rd(1)} = 80.49$	80.49	Front plate - tension
$F_{t,wb,Rd(1)} = 204.63$	204.63	Beam web - tension
$B_{p,Rd} = 312.69$	312.69	Bolts due to shear punching
$V_{wp,Rd}/\beta = 190.77$	190.77	Web panel - shear
$F_{c,wc,Rd} = 507.27$	507.27	Column web - compression
$F_{c,fb,Rd} = 374.32$	374.32	Beam flange - compression

### RESISTANCE OF THE BOLT ROW NO. 2

<b>F<sub>t2,Rd,comp</sub> - Formula</b>	<b>F<sub>t2,Rd,comp</sub></b>	<b>Component</b>
$F_{t2,Rd} = \text{Min} (F_{t2,Rd,comp})$	21.14	Bolt row resistance
$F_{t,fc,Rd(2)} = 133.26$	133.26	Column flange - tension
$F_{t,wc,Rd(2)} = 152.33$	152.33	Column web - tension
$F_{t,ep,Rd(2)} = 78.07$	78.07	Front plate - tension
$F_{t,wb,Rd(2)} = 198.48$	198.48	Beam web - tension
$B_{p,Rd} = 312.69$	312.69	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^1 F_{ti,Rd} = 190.77 - 80.49$	110.28	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{ij,Rd} = 507.27 - 80.49$	426.78	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{ij,Rd} = 374.32 - 80.49$	293.83	Beam flange - compression
$F_{t,fc,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 190.81 - 80.49$	110.31	Column flange - tension - group
$F_{t,wc,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 193.25 - 80.49$	112.76	Column web - tension - group
$F_{t,ep,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 101.63 - 80.49$	21.14	Front plate - tension - group
$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{ij,Rd} = 258.38 - 80.49$	177.89	Beam web - tension - group

### RESISTANCE OF THE BOLT ROW NO. 3

<b>F<sub>t3,Rd,comp</sub> - Formula</b>	<b>F<sub>t3,Rd,comp</sub></b>	<b>Component</b>
$F_{t3,Rd} = \text{Min} (F_{t3,Rd,comp})$	59.09	Bolt row resistance
$F_{t,fc,Rd(3)} = 133.26$	133.26	Column flange - tension
$F_{t,wc,Rd(3)} = 152.33$	152.33	Column web - tension
$F_{t,ep,Rd(3)} = 78.07$	78.07	Front plate - tension
$F_{t,wb,Rd(3)} = 198.48$	198.48	Beam web - tension

<b>F<sub>t3,Rd,comp</sub> - Formula</b>	<b>F<sub>t3,Rd,comp</sub></b>	<b>Component</b>
$B_{p,Rd} = 312.69$	312.69	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum 1^2 F_{tj,Rd} = 190.77 - 101.63$	89.14	Web panel - shear
$F_{c,wc,Rd} - \sum 1^2 F_{tj,Rd} = 507.27 - 101.63$	405.63	Column web - compression
$F_{c,fb,Rd} - \sum 1^2 F_{tj,Rd} = 374.32 - 101.63$	272.69	Beam flange - compression
$F_{t,fc,Rd(3+2)} - \sum 2^2 F_{tj,Rd} = 207.56 - 21.14$	186.42	Column flange - tension - group
$F_{t,wc,Rd(3+2)} - \sum 2^2 F_{tj,Rd} = 202.57 - 21.14$	181.43	Column web - tension - group
$F_{t,fc,Rd(3+2+1)} - \sum 2^1 F_{tj,Rd} = 324.13 - 101.63$	222.50	Column flange - tension - group
$F_{t,wc,Rd(3+2+1)} - \sum 2^1 F_{tj,Rd} = 245.26 - 101.63$	143.63	Column web - tension - group
$F_{t,ep,Rd(3+2)} - \sum 2^2 F_{tj,Rd} = 99.21 - 21.14$	78.07	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum 2^2 F_{tj,Rd} = 252.22 - 21.14$	231.08	Beam web - tension - group
$F_{t,ep,Rd(3+2+1)} - \sum 2^1 F_{tj,Rd} = 160.73 - 101.63$	59.09	Front plate - tension - group
$F_{t,wb,Rd(3+2+1)} - \sum 2^1 F_{tj,Rd} = 408.61 - 101.63$	306.98	Beam web - tension - group

### SUMMARY TABLE OF FORCES

Nr	h <sub>j</sub>	F <sub>tj,Rd</sub>	F <sub>t,fc,Rd</sub>	F <sub>t,wc,Rd</sub>	F <sub>t,ep,Rd</sub>	F <sub>t,wb,Rd</sub>	F <sub>t,Rd</sub>	B <sub>p,Rd</sub>
1	178	80.49	133.26	152.33	80.49	204.63	508.32	312.69
2	108	21.14	133.26	152.33	78.07	198.48	508.32	312.69
3	38	59.09	133.26	152.33	78.07	198.48	508.32	312.69

### CONNECTION RESISTANCE FOR BENDING M<sub>j,Rd</sub>

$$M_{j,Rd} = \sum h_j F_{tj,Rd}$$

$$M_{j,Rd} = 18.82 \text{ [kN*m]} \text{ Connection resistance for bending} \quad [6.2]$$

$$M_{b1,Ed} / M_{j,Rd} \leq 1,0 \quad 0.63 < 1.00 \quad \text{verified} \quad (0.63)$$

### CONNECTION RESISTANCE FOR SHEAR

$$\alpha_v = 0.60 \quad \text{Coefficient for calculation of } F_{v,Rd} \quad [\text{Table 3.4}]$$

$$F_{v,Rd} = 217.15 \text{ [kN]} \quad \text{Shear resistance of a single bolt} \quad [\text{Table 3.4}]$$

$$F_{t,Rd,max} = 254.16 \text{ [kN]} \quad \text{Tensile resistance of a single bolt} \quad [\text{Table 3.4}]$$

$$F_{b,Rd,int} = 74.08 \text{ [kN]} \quad \text{Bearing resistance of an intermediate bolt} \quad [\text{Table 3.4}]$$

$$F_{b,Rd,ext} = 54.87 \text{ [kN]} \quad \text{Bearing resistance of an outermost bolt} \quad [\text{Table 3.4}]$$

Nr	F <sub>tj,Rd,N</sub>	F <sub>tj,Ed,N</sub>	F <sub>tj,Rd,M</sub>	F <sub>tj,Ed,M</sub>	F <sub>tj,Ed</sub>	F <sub>vj,Rd</sub>
1	508.32	-13.95	80.49	50.46	36.51	148.16
2	508.32	-13.95	21.14	13.25	-0.70	148.16
3	508.32	-13.95	59.09	37.04	23.09	148.16

F<sub>tj,Rd,N</sub> – Bolt row resistance for simple tension

F<sub>tj,Ed,N</sub> – Force due to axial force in a bolt row

F<sub>tj,Rd,M</sub> – Bolt row resistance for simple bending

F<sub>tj,Ed,M</sub> – Force due to moment in a bolt row

F<sub>tj,Ed</sub> – Maximum tensile force in a bolt row

F<sub>vj,Rd</sub> – Reduced bolt row resistance

$$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$$

$$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$$

$$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$$

$$F_{vj,Rd} = \text{Min} (\eta_h F_{v,Rd} (1 - F_{tj,Ed} / (1.4 \eta_h F_{t,Rd,max})), \eta_h F_{v,Rd}, \eta_h F_{b,Rd})$$

$$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd} \quad \text{[Table 3.4]}$$

$$V_{j,Rd} = 444.48 \quad \text{[kN]} \quad \text{Connection resistance for shear} \quad \text{[Table 3.4]}$$

$$V_{b1,Ed} / V_{j,Rd} \leq 1,0 \quad 0.02 < 1.00 \quad \text{verified} \quad (0.02)$$

## WELD RESISTANCE

$$A_w = 54.05 \quad \text{[cm}^2\text{]} \quad \text{Area of all welds} \quad \text{[4.5.3.2(2)]}$$

$$A_{wy} = 35.01 \quad \text{[cm}^2\text{]} \quad \text{Area of horizontal welds} \quad \text{[4.5.3.2(2)]}$$

$$A_{wz} = 19.04 \quad \text{[cm}^2\text{]} \quad \text{Area of vertical welds} \quad \text{[4.5.3.2(2)]}$$

$$I_{wy} = 5312.16 \quad \text{[cm}^4\text{]} \quad \text{Moment of inertia of the weld arrangement with respect to the hor. axis} \quad \text{[4.5.3.2(5)]}$$

$$\sigma_{\perp \max} = \tau_{\perp \max} = 27219.93 \quad \text{[kPa]} \quad \text{Normal stress in a weld} \quad \text{[4.5.3.2(5)]}$$

$$\sigma_{\perp} = \tau_{\perp} = 22696.28 \quad \text{[kPa]} \quad \text{Stress in a vertical weld} \quad \text{[4.5.3.2(5)]}$$

$$\tau_{\parallel} = -4191.18 \quad \text{[kPa]} \quad \text{Tangent stress} \quad \text{[4.5.3.2(5)]}$$

$$\beta_w = 0.80 \quad \text{Correlation coefficient} \quad \text{[4.5.3.2(7)]}$$

$$\sqrt{[\sigma_{\perp \max}^2 + 3 * (\tau_{\perp \max}^2)]} \leq f_u / (\beta_w * \gamma_{M2}) \quad 54439.86 < 360000.00 \quad \text{verified} \quad (0.15)$$

$$\sqrt{[\sigma_{\perp}^2 + 3 * (\tau_{\perp}^2 + \tau_{\parallel}^2)]} \leq f_u / (\beta_w * \gamma_{M2}) \quad 45969.37 < 360000.00 \quad \text{verified} \quad (0.13)$$

$$\sigma_{\perp} \leq 0.9 * f_u / \gamma_{M2} \quad 27219.93 < 259200.00 \quad \text{verified} \quad (0.11)$$

## CONNECTION STIFFNESS

$$t_{wash} = 5 \quad \text{[mm]} \quad \text{Washer thickness} \quad \text{[6.2.6.3.(2)]}$$

$$h_{head} = 17 \quad \text{[mm]} \quad \text{Bolt head height} \quad \text{[6.2.6.3.(2)]}$$

$$h_{nut} = 24 \quad \text{[mm]} \quad \text{Bolt nut height} \quad \text{[6.2.6.3.(2)]}$$

$$L_b = 48 \quad \text{[mm]} \quad \text{Bolt length} \quad \text{[6.2.6.3.(2)]}$$

$$k_{10} = 12 \quad \text{[mm]} \quad \text{Stiffness coefficient of bolts} \quad \text{[6.3.2.(1)]}$$

## STIFFNESSES OF BOLT ROWS

Nr	h <sub>j</sub>	k <sub>3</sub>	k <sub>4</sub>	k <sub>5</sub>	k <sub>eff,j</sub>	k <sub>eff,j</sub> h <sub>j</sub>	k <sub>eff,j</sub> h <sub>j</sub> <sup>2</sup>
					Sum	3.31	46.75
1	178	3	11	3	1	2.03	36.17
2	108	2	7	2	1	0.82	8.87
3	38	3	12	3	1	0.45	1.71

$$k_{eff,j} = 1 / (\sum_3^5 (1 / k_{i,j})) \quad \text{[6.3.3.1.(2)]}$$

$$Z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$$

$$Z_{eq} = 141 \quad \text{[mm]} \quad \text{Equivalent force arm} \quad \text{[6.3.3.1.(3)]}$$

$$k_{eq} = \sum_j k_{eff,j} h_j / Z_{eq}$$

$$k_{eq} = 2 \quad \text{[mm]} \quad \text{Equivalent stiffness coefficient of a bolt arrangement} \quad \text{[6.3.3.1.(1)]}$$

$$A_{vc} = 14.47 \quad \text{[cm}^2\text{]} \quad \text{Shear area} \quad \text{EN1993-1-1:[6.2.6.(3)]}$$

$$\beta = 1.00 \quad \text{Transformation parameter} \quad \text{[5.3.(7)]}$$

$$z = 141 \quad \text{[mm]} \quad \text{Lever arm} \quad \text{[6.2.5]}$$

$$k_1 = 4 \quad \text{[mm]} \quad \text{Stiffness coefficient of the column web panel subjected to shear} \quad \text{[6.3.2.(1)]}$$

$$k_2 = \infty \quad \text{Stiffness coefficient of the compressed column web} \quad \text{[6.3.2.(1)]}$$

$$S_{j,ini} = E z_{eq}^2 / \sum_i (1 / k_1 + 1 / k_2 + 1 / k_{eq}) \quad [6.3.1.(4)]$$

$$S_{j,ini} = 6129.19 \text{ [kN*m]} \text{ Initial rotational stiffness} \quad [6.3.1.(4)]$$

$$\mu = 1.00 \text{ Stiffness coefficient of a connection} \quad [6.3.1.(6)]$$

$$S_j = S_{j,ini} / \mu \quad [6.3.1.(4)]$$

$$S_j = 6129.19 \text{ [kN*m]} \text{ Final rotational stiffness} \quad [6.3.1.(4)]$$

**Connection classification due to stiffness.**

$$S_{j,rig} = 13075.88 \text{ [kN*m]} \text{ Stiffness of a rigid connection} \quad [5.2.2.5]$$

$$S_{j,pin} = 817.24 \text{ [kN*m]} \text{ Stiffness of a pinned connection} \quad [5.2.2.5]$$

$S_{j,pin} \leq S_{j,ini} < S_{j,rig}$  SEMI-RIGID

**WEAKEST COMPONENT:**

**REMARKS**

---

Distance of bolts from an edge is too large. 127 [mm] > 72 [mm]

Distance of bolts from an edge is too small. 25 [mm] < 31 [mm]

<b>Connection conforms to the code</b>	Ratio	0.63
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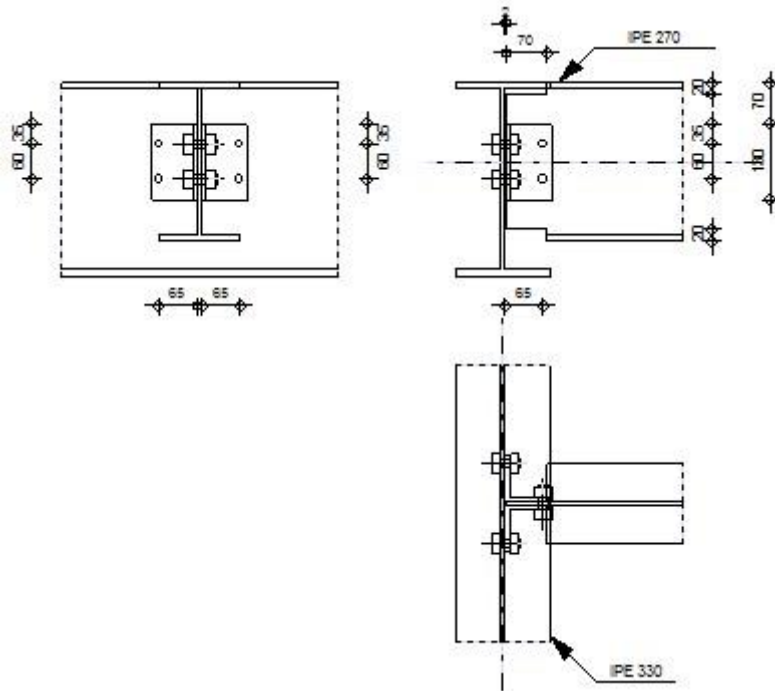


## Calculation of the beam-to-beam (web) connection

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



Ratio  
0.55



### GENERAL

Connection no.: 2  
Connection name: Beam-beam (web)

### GEOMETRY

#### PRINCIPAL BEAM

Section: IPE 330

$\alpha = -90.0$  [Deg] Inclination angle  
 $h_g = 330$  [mm] Height of the principal beam section  
 $b_{fg} = 160$  [mm] Width of the flange of the principal beam section  
 $t_{wg} = 8$  [mm] Thickness of the web of the principal beam section  
 $t_{fg} = 12$  [mm] Thickness of the flange of the principal beam section  
 $r_g = 18$  [mm] Fillet radius of the web of the principal beam section  
 $A_p = 62.61$  [cm<sup>2</sup>] Cross-sectional area of a principal beam  
 $I_{yp} = 11766.90$  [cm<sup>4</sup>] Moment of inertia of the principal beam section

Material: S 235

$f_{yg} = 255000.00$  [kPa] Design resistance  
 $f_{ug} = 360000.00$  [kPa] Tensile resistance

#### BEAM

Section: IPE 270

$\alpha = 0.0$  [Deg] Inclination angle

$h_b = 270$  [mm] Height of beam section

$b_b = 135$  [mm] Width of beam section

$t_{wb} = 7$  [mm] Thickness of the web of beam section

$t_{fb} = 10$  [mm] Thickness of the flange of beam section

$r_b = 15$  [mm] Radius of beam section fillet

$A_b = 45.94$  [cm<sup>2</sup>] Cross-sectional area of a beam

$I_{yb} = 5789.78$  [cm<sup>4</sup>] Moment of inertia of the beam section

Material: S 235

$f_{yb} = 255000.00$  [kPa] Design resistance

$f_{ub} = 360000.00$  [kPa] Tensile resistance

## **BEAM CUT**

$h_1 = 20$  [mm] Top cut-out

$h_2 = 20$  [mm] Bottom cut-out

$l = 70$  [mm] Cut-out length

## **ANGLE**

Section: CAE 80x8

$h_k = 80$  [mm] Height of angle section

$b_k = 80$  [mm] Width of angle section

$t_{fk} = 8$  [mm] Thickness of the flange of angle section

$r_k = 10$  [mm] Fillet radius of the web of angle section

$l_k = 130$  [mm] Angle length

Material: S 235

$f_{yk} = 255000.00$  [kPa] Design resistance

$f_{uk} = 360000.00$  [kPa] Tensile resistance

## **BOLTS**

### **BOLTS CONNECTING ANGLE WITH PRINCIPAL BEAM**

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

Class = 4.6 Bolt class

$d = 16$  [mm] Bolt diameter

$d_0 = 18$  [mm] Bolt opening diameter

$A_s = 1.57$  [cm<sup>2</sup>] Effective section area of a bolt

$A_v = 2.01$  [cm<sup>2</sup>] Area of bolt section

$f_{ub} = 400000.00$  [kPa] Tensile resistance

$k = 1$  Number of bolt columns

$w = 2$  Number of bolt rows

$e_1 = 35$  [mm] Level of first bolt

$p_1 = 60$  [mm] Vertical spacing

### **BOLTS CONNECTING ANGLE WITH BEAM**

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

Class = 4.6 Bolt class

$d = 16$  [mm] Bolt diameter

$d_0 = 18$  [mm] Bolt opening diameter

$A_s = 1.57$  [cm<sup>2</sup>] Effective section area of a bolt

$A_v = 2.01$  [cm<sup>2</sup>] Area of bolt section

$f_{ub} = 400000.00$  [kPa] Tensile resistance

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

Class =	4.6	Bolt class
k =	1	Number of bolt columns
w =	2	Number of bolt rows
e <sub>1</sub> =	35 [mm]	Level of first bolt
p <sub>1</sub> =	60 [mm]	Vertical spacing

## **MATERIAL FACTORS**

γ <sub>M0</sub> =	1.00	Partial safety factor	[2.2]
γ <sub>M2</sub> =	1.25	Partial safety factor	[2.2]

## **LOADS**

Case: Manual calculations.

N <sub>b,Ed</sub> =	0.00 [kN]	Axial force
V <sub>b,Ed</sub> =	22.00 [kN]	Shear force
M <sub>b,Ed</sub> =	0.00 [kN*m]	Bending moment

## **RESULTS**

### **BOLTS CONNECTING ANGLE WITH PRINCIPAL BEAM**

#### **BOLT CAPACITIES**

F <sub>v,Rd</sub> =	38.60 [kN]	Shear resistance of the shank of a single bolt	F <sub>v,Rd</sub> = 0.6*f <sub>ub</sub> *A <sub>v</sub> *m/γ <sub>M2</sub>
F <sub>t,Rd</sub> =	45.22 [kN]	Tensile resistance of a single bolt	F <sub>t,Rd</sub> = 0.9*f <sub>u</sub> *A <sub>s</sub> /γ <sub>M2</sub>

#### **Bolt bearing on the principal beam web**

Direction x

k <sub>1x</sub> =	2.50	Coefficient for calculation of F <sub>b,Rd</sub>	k <sub>1x</sub> = min[2.8*(e <sub>1</sub> /d <sub>0</sub> )-1.7, 1.4*(p <sub>1</sub> /d <sub>0</sub> )-1.7, 2.5]
k <sub>1x</sub> > 0.0	2.50 > 0.00	verified	
α <sub>bx</sub> =	1.00	Coefficient for calculation of F <sub>b,Rd</sub>	α <sub>bx</sub> = min[e <sub>2</sub> /(3*d <sub>0</sub> ), f <sub>ub</sub> /f <sub>u</sub> , 1]
α <sub>bx</sub> > 0.0	1.00 > 0.00	verified	
F <sub>b,Rd1x</sub> =	86.40 [kN]	Bearing resistance of a single bolt	F <sub>b,Rd1x</sub> = k <sub>1x</sub> *α <sub>bx</sub> *f <sub>u</sub> *d*t/γ <sub>M2</sub>

Direction z

k <sub>1z</sub> =	2.50	Coefficient for calculation of F <sub>b,Rd</sub>	k <sub>1z</sub> = min[2.8*(e <sub>2</sub> /d <sub>0</sub> )-1.7, 2.5]
k <sub>1z</sub> > 0.0	2.50 > 0.00	verified	
α <sub>bz</sub> =	0.86	Coefficient for calculation of F <sub>b,Rd</sub>	α <sub>bz</sub> = min[e <sub>1</sub> /(3*d <sub>0</sub> ), p <sub>1</sub> /(3*d <sub>0</sub> )-0.25, f <sub>ub</sub> /f <sub>u</sub> , 1]
α <sub>bz</sub> > 0.0	0.86 > 0.00	verified	
F <sub>b,Rd1z</sub> =	74.40 [kN]	Bearing resistance of a single bolt	F <sub>b,Rd1z</sub> = k <sub>1z</sub> *α <sub>bz</sub> *f <sub>u</sub> *d*t/γ <sub>M2</sub>

#### **Bolt bearing on the angle**

Direction x

k <sub>1x</sub> =	2.50	Coefficient for calculation of F <sub>b,Rd</sub>	k <sub>1x</sub> = min[2.8*(e <sub>1</sub> /d <sub>0</sub> )-1.7, 1.4*(p <sub>1</sub> /d <sub>0</sub> )-1.7, 2.5]
k <sub>1x</sub> > 0.0	2.50 > 0.00	verified	
α <sub>bx</sub> =	0.28	Coefficient for calculation of F <sub>b,Rd</sub>	α <sub>bx</sub> = min[e <sub>2</sub> /(3*d <sub>0</sub> ), f <sub>ub</sub> /f <sub>u</sub> , 1]
α <sub>bx</sub> > 0.0	0.28 > 0.00	verified	

$$F_{b,Rd2x} = 25.60 \quad [\text{kN}] \quad \text{Bearing resistance of a single bolt} \quad F_{b,Rd2x} = k_{1x} \cdot \alpha_{bx} \cdot f_u \cdot d \cdot t_i / \gamma_{M2}$$

Direction z

$$k_{1z} = 0.63 \quad \text{Coefficient for calculation of } F_{b,Rd} \quad k_{1z} = \min[2.8 \cdot (e_2/d_0) - 1.7, 2.5]$$

$$k_{1z} > 0.0 \quad 0.63 > 0.00 \quad \text{verified}$$

$$\alpha_{bz} = 0.65 \quad \text{Coefficient for calculation of } F_{b,Rd} \quad \alpha_{bz} = \min[e_1/(3 \cdot d_0), p_1/(3 \cdot d_0) - 0.25, f_{ub}/f_u, 1]$$

$$\alpha_{bz} > 0.0 \quad 0.65 > 0.00 \quad \text{verified}$$

$$F_{b,Rd2z} = 15.13 \quad [\text{kN}] \quad \text{Bearing resistance of a single bolt} \quad F_{b,Rd2z} = k_{1z} \cdot \alpha_{bz} \cdot f_u \cdot d \cdot t_i / \gamma_{M2}$$

## FORCES ACTING ON BOLTS IN THE PRINCIPAL BEAM - ANGLE CONNECTION

### Bolt shear

$$e = 68 \quad [\text{mm}] \quad \text{Distance between centroid of a bolt group of an angle and center of the beam web}$$

$$M_0 = \frac{0.7}{5} \quad \frac{[\text{kN} \cdot \text{m}]}{\text{m}} \quad \text{Real bending moment} \quad M_0 = 0.5 \cdot V_{b,Ed} \cdot e$$

$$F_{Vz} = \frac{5.5}{0} \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the shear force} \quad F_{Vz} = 0.5 \cdot |V_{b,Ed}| / n$$

$$F_{Mx} = \frac{12.}{52} \quad [\text{kN}] \quad \text{Component force in a bolt due to influence of the moment} \quad F_{Mx} = |M_0| \cdot z_i / \sum z_i^2$$

$$F_{x,Ed} = \frac{12.}{52} \quad [\text{kN}] \quad \text{Design total force in a bolt on the direction x} \quad F_{x,Ed} = F_{Nx} + F_{Mx}$$

$$F_{z,Ed} = \frac{5.5}{0} \quad [\text{kN}] \quad \text{Design total force in a bolt on the direction z} \quad F_{z,Ed} = F_{Vz} + F_{Mz}$$

$$F_{Rdx} = \frac{25.}{60} \quad [\text{kN}] \quad \text{Effective design capacity of a bolt on the direction x} \quad F_{Rdx} = \min(F_{vRd}, F_{bRd1x}, F_{bRd2x})$$

$$F_{Rdz} = \frac{15.}{13} \quad [\text{kN}] \quad \text{Effective design capacity of a bolt on the direction z} \quad F_{Rdz} = \min(F_{vRd}, F_{bRd1z}, F_{bRd2z})$$

$$|F_{x,Ed}| \leq F_{Rdx} \quad |12.52| < 25.60 \quad \text{verified} \quad (0.49)$$

$$|F_{z,Ed}| \leq F_{Rdz} \quad |5.50| < 15.13 \quad \text{verified} \quad (0.36)$$

### Bolt tension

$$e = 69 \quad [\text{mm}] \quad \text{Distance between centroid of a bolt group and center of the principal beam web}$$

$$M_{0t} = \frac{0.7}{6} \quad \frac{[\text{kN} \cdot \text{m}]}{\text{m}} \quad \text{Real bending moment} \quad M_{0t} = 0.5 \cdot V_{b,Ed} \cdot e$$

$$F_{t,Ed} = \frac{12.}{60} \quad [\text{kN}] \quad \text{Tensile force in the outermost bolt} \quad F_{t,Ed} = M_{0t} \cdot z_{\max} / \sum z_i^2 + 0.5 \cdot N_{b2,Ed} / n$$

$$F_{t,Ed} \leq F_{t,Rd} \quad 12.60 < 45.22 \quad \text{verified} \quad (0.28)$$

### Simultaneous action of a tensile force and a shear force in a bolt

$$F_{V,Ed} = 13.68 \quad [\text{kN}] \quad \text{Resultant shear force in a bolt} \quad F_{V,Ed} = \sqrt{F_{x,Ed}^2 + F_{z,Ed}^2}$$

$$F_{V,Ed} / F_{v,Rd} + F_{t,Ed} / (1.4 \cdot F_{t,Rd}) \leq 1.0 \quad 0.55 < 1.00 \quad \text{verified} \quad (0.55)$$

## BOLTS CONNECTING ANGLE WITH BEAM

### BOLT CAPACITIES

$$F_{v,Rd} = 77.21 \quad [\text{kN}] \quad \text{Shear resistance of the shank of a single bolt} \quad F_{v,Rd} = 0.6 \cdot f_{ub} \cdot A_v \cdot m / \gamma_{M2}$$

### Bolt bearing on the beam

Direction x

$$k_{1x} = 2.50 \quad \text{Coefficient for calculation of } F_{b,Rd} \quad k_{1x} = \min[2.8 \cdot (e_1/d_0) - 1.7, 1.4 \cdot (p_1/d_0) - 1.7, 2.5]$$

$$k_{1x} > 0.0 \quad 2.50 > 0.00 \quad \text{verified}$$

$\alpha_{bx} = 1.00$	Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bx} = \min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$	$1.00 > 0.00$	verified
$F_{b,Rd1x} = 76.03$ [kN]	Bearing resistance of a single bolt	$F_{b,Rd1x} = k_{1x} * \alpha_{bx} * f_u * d * t_i / \gamma_{M2}$

Direction z

$k_{1z} = 2.50$	Coefficient for calculation of $F_{b,Rd}$	$k_{1z} = \min[2.8*(e_2/d_0) - 1.7, 2.5]$
$k_{1z} > 0.0$	$2.50 > 0.00$	verified
$\alpha_{bz} = 0.86$	Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bz} = \min[e_1/(3*d_0), p_1/(3*d_0) - 0.25, f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$	$0.86 > 0.00$	verified
$F_{b,Rd1z} = 65.47$ [kN]	Bearing resistance of a single bolt	$F_{b,Rd1z} = k_{1z} * \alpha_{bz} * f_u * d * t_i / \gamma_{M2}$

### Bolt bearing on the angle

Direction x

$k_{1x} = 2.50$	Coefficient for calculation of $F_{b,Rd}$	$k_{1x} = \min[2.8*(e_1/d_0) - 1.7, 1.4*(p_1/d_0) - 1.7, 2.5]$
$k_{1x} > 0.0$	$2.50 > 0.00$	verified
$\alpha_{bx} = 0.28$	Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bx} = \min[e_2/(3*d_0), f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$	$0.28 > 0.00$	verified
$F_{b,Rd2x} = 51.20$ [kN]	Bearing resistance of a single bolt	$F_{b,Rd2x} = k_{1x} * \alpha_{bx} * f_u * d * t_i / \gamma_{M2}$

Direction z

$k_{1z} = 0.63$	Coefficient for calculation of $F_{b,Rd}$	$k_{1z} = \min[2.8*(e_2/d_0) - 1.7, 2.5]$
$k_{1z} > 0.0$	$0.63 > 0.00$	verified
$\alpha_{bz} = 0.65$	Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bz} = \min[e_1/(3*d_0), p_1/(3*d_0) - 0.25, f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$	$0.65 > 0.00$	verified
$F_{b,Rd2z} = 30.26$ [kN]	Bearing resistance of a single bolt	$F_{b,Rd2z} = k_{1z} * \alpha_{bz} * f_u * d * t_i / \gamma_{M2}$

### FORCES ACTING ON BOLTS IN THE ANGLE - BEAM CONNECTION

#### Bolt shear

$e = 69$ [mm]	Distance between centroid of a bolt group and center of the principal beam web	
$M_0 = 1.5$ [kN*m]	Real bending moment	$M_0 = M_{b,Ed} + V_{b,Ed} * e$
$F_{Vz} = 11.00$ [kN]	Component force in a bolt due to influence of the shear force	$F_{Vz} =  V_{b,Ed}  / n$
$F_{Mx} = 25.21$ [kN]	Component force in a bolt due to influence of the moment	$F_{Mx} =  M_0  * z_i / \sum z_i^2$
$F_{x,Ed} = 25.21$ [kN]	Design total force in a bolt on the direction x	$F_{x,Ed} = F_{Nx} + F_{Mx}$
$F_{z,Ed} = 11.00$ [kN]	Design total force in a bolt on the direction z	$F_{z,Ed} = F_{Vz} + F_{Mz}$
$F_{Rdx} = 51.20$ [kN]	Effective design capacity of a bolt on the direction x	$F_{Rdx} = \min(F_{VRd}, F_{bRd1x}, F_{bRd2x})$
$F_{Rdz} = 30.26$ [kN]	Effective design capacity of a bolt on the direction z	$F_{Rdz} = \min(F_{VRd}, F_{bRd1z}, F_{bRd2z})$
$ F_{x,Ed}  \leq F_{Rdx}$	$ 25.21  < 51.20$	verified (0.49)
$ F_{z,Ed}  \leq F_{Rdz}$	$ 11.00  < 30.26$	verified (0.36)

### VERIFICATION OF THE SECTION DUE TO BLOCK TEARING

#### ANGLE

$A_{nt} = 0.48$ [cm <sup>2</sup> ]	Net area of the section in tension
$A_{nv} = 5.44$ [cm <sup>2</sup> ]	Area of the section in shear

## ANGLE

$$A_{nt} = 0.48 \text{ [cm}^2\text{]} \quad \text{Net area of the section in tension}$$
$$V_{effRd} = 87.0 \text{ [kN]} \quad \text{Design capacity of a section weakened by openings}$$
$$V_{effRd} = 0.5 \cdot f_u \cdot A_{nt} / \gamma_{M2} + (1/\sqrt{3}) \cdot f_y \cdot A_{nv} / \gamma_{M0} \quad (0.13)$$
$$|0.5 \cdot V_{b,Ed}| \leq V_{effRd} \quad |11.00| < 87.00 \quad \text{verified}$$

## BEAM

$$A_{nt} = 3.56 \text{ [cm}^2\text{]} \quad \text{Net area of the section in tension}$$
$$A_{nv} = 7.79 \text{ [cm}^2\text{]} \quad \text{Area of the section in shear}$$
$$V_{effRd} = 165.9 \text{ [kN]} \quad \text{Design capacity of a section weakened by openings}$$
$$V_{effRd} = 0.5 \cdot f_u \cdot A_{nt} / \gamma_{M2} + (1/\sqrt{3}) \cdot f_y \cdot A_{nv} / \gamma_{M0} \quad (0.13)$$
$$|V_{b,Ed}| \leq V_{effRd} \quad |22.00| < 165.98 \quad \text{verified}$$

## REMARKS

Length of the beam cut-out is too small	70 [mm] < 74 [mm]
Distance between bolt and angle vertical edge in the beam is too small	15 [mm] < 22 [mm]
Distance between bolt and vertical edge of an angle on the principal beam is too small	15 [mm] < 22 [mm]

**Connection conforms to the code**

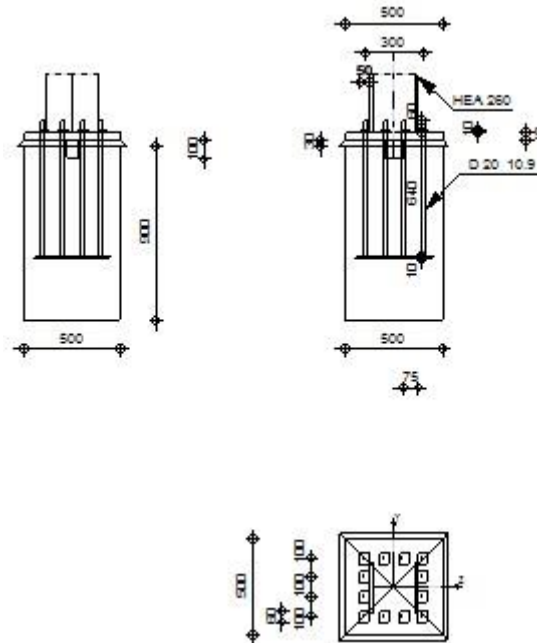
**Ratio** 0.55



## Fixed column base design

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

Ratio  
**0.85**



## GENERAL

Connection no.: 3  
Connection name: Fixed column base

## GEOMETRY

### COLUMN

Section: HEA 260

$L_c =$	5.00	[m]	Column length
$\alpha =$	0.0	[Deg]	Inclination angle
$h_c =$	250	[mm]	Height of column section
$b_{fc} =$	260	[mm]	Width of column section
$t_{wc} =$	8	[mm]	Thickness of the web of column section
$t_{fc} =$	13	[mm]	Thickness of the flange of column section
$r_c =$	24	[mm]	Radius of column section fillet
$A_c =$	86.82	[cm <sup>2</sup> ]	Cross-sectional area of a column
$I_{yc} =$	10455.00	[cm <sup>4</sup> ]	Moment of inertia of the column section

Material: S 235

$f_{yc} =$	235000.00	[kPa]	Resistance
$f_{uc} =$	360000.00	[kPa]	Yield strength of a material

### COLUMN BASE

$l_{pd} =$	500	[mm]	Length
$b_{pd} =$	500	[mm]	Width
$t_{pd} =$	40	[mm]	Thickness

Material: S 235  
 $f_{ypd} = 235000.00$  [kPa] Resistance  
 $f_{upd} = 360000.00$  [kPa] Yield strength of a material

## **ANCHORAGE**

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

Class = 10.9 Anchor class  
 $f_{yb} = 900000.00$  [kPa] Yield strength of the anchor material  
 $f_{ub} = 1000000.00$  [kPa] Tensile strength of the anchor material  
 $d = 20$  [mm] Bolt diameter  
 $A_s = 2.45$  [cm<sup>2</sup>] Effective section area of a bolt  
 $A_v = 3.14$  [cm<sup>2</sup>] Area of bolt section  
 $n_H = 4$  Number of bolt columns  
 $n_V = 4$  Number of bolt rows  
Horizontal spacing  $e_{Hi} = 100;100$  [mm]  
Vertical spacing  $e_{Vi} = 100;100$  [mm]

### **Anchor dimensions**

$L_1 = 60$  [mm]  
 $L_2 = 640$  [mm]  
 $L_3 = 0$  [mm]

### **Anchor plate**

$l_p = 100$  [mm] Length  
 $b_p = 100$  [mm] Width  
 $t_p = 10$  [mm] Thickness  
Material: S 235  
 $f_y = 235000.00$  [kPa] Resistance

### **Washer**

$l_{wd} = 50$  [mm] Length  
 $b_{wd} = 60$  [mm] Width  
 $t_{wd} = 10$  [mm] Thickness

## **WEDGE**

Section: IPE 100  
 $l_w = 100$  [mm] Length  
Material: S 235  
 $f_{yw} = 235000.00$  [kPa] Resistance

## **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**

$L = 500$  [mm] Spread footing length  
 $B = 500$  [mm] Spread footing width  
 $H = 900$  [mm] Spread footing height

### **Concrete**

Class Usuario  
 $f_{ck} = 382500.00$  [kPa] Characteristic resistance for compression

### Grout layer

$t_g =$	30	[mm]	Thickness of leveling layer (grout)
$f_{ck,g} =$	12000.00	[kPa]	Characteristic resistance for compression
$C_{f,d} =$	0.30		Coeff. of friction between the base plate and concrete

### WELDS

$a_p =$	6	[mm]	Footing plate of the column base
$a_w =$	6	[mm]	Wedge

### LOADS

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Case: Manual calculations.

$N_{j,Ed} =$	-224.00	[kN]	Axial force
$V_{j,Ed,y} =$	5.00	[kN]	Shear force
$V_{j,Ed,z} =$	-28.00	[kN]	Shear force
$M_{j,Ed,y} =$	-50.00	[kN*m]	Bending moment
$M_{j,Ed,z} =$	9.00	[kN*m]	Bending moment

### RESULTS

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### COMPRESSION ZONE

#### COMPRESSION OF CONCRETE

$f_{cd} =$	255000.00	[kPa]	Design compressive resistance	EN 1992-1:[3.1.6.(1)]
$f_j =$	170000.00	[kPa]	Design bearing resistance under the base plate	[6.2.5.(7)]
$c =$	$t_p \sqrt{(f_{yp}/(3*f_j*\gamma_{Mo}))}$			
$c =$	27	[mm]	Additional width of the bearing pressure zone	[6.2.5.(4)]
$b_{eff} =$	67	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
$l_{eff} =$	314	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]
$A_{c0} =$	209.97	[cm <sup>2</sup> ]	Area of the joint between the base plate and the foundation	EN 1992-1:[6.7.(3)]
$A_{c1} =$	1002.07	[cm <sup>2</sup> ]	Maximum design area of load distribution	EN 1992-1:[6.7.(3)]
$F_{rd,u} =$	$A_{c0}*f_{cd}*\sqrt{(A_{c1}/A_{c0})} \leq 3*A_{c0}*f_{cd}$			
$A_{c1} =$	1002.07	[cm <sup>2</sup> ]	Maximum design area of load distribution	EN 1992-1:[6.7.(3)]
$\beta_j =$	0.67		Reduction factor for compression	[6.2.5.(7)]
$f_{jd} =$	$\beta_j * F_{rd,u} / (b_{eff} * l_{eff})$			
$f_{jd} =$	371380.41	[kPa]	Design bearing resistance	[6.2.5.(7)]
$A_{c,n} =$	525.44	[cm <sup>2</sup> ]	Bearing area for compression	[6.2.8.2.(1)]
$A_{c,y} =$	209.97	[cm <sup>2</sup> ]	Bearing area for bending My	[6.2.8.3.(1)]
$A_{c,z} =$	209.97	[cm <sup>2</sup> ]	Bearing area for bending Mz	[6.2.8.3.(1)]
$F_{c,Rd,i} =$	$A_{c,i} * f_{jd}$			
$F_{c,Rd,n} =$	19513.81	[kN]	Bearing resistance of concrete for compression	[6.2.8.2.(1)]
$F_{c,Rd,y} =$	7797.91	[kN]	Bearing resistance of concrete for bending My	[6.2.8.3.(1)]
$F_{c,Rd,z} =$	7797.91	[kN]	Bearing resistance of concrete for bending Mz	[6.2.8.3.(1)]

#### COLUMN FLANGE AND WEB IN COMPRESSION

$CL =$	2.00		Section class	EN 1993-1-1:[5.5.2]
$W_{pl,y} =$	919.86	[cm <sup>3</sup> ]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]
$M_{c,Rd,y} =$	216.17	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
$h_{f,y} =$	238	[mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]

CL = 2.00	Section class	EN 1993-1-1:[5.5.2]
$F_{c,fc,Rd,y} = M_{c,Rd,y} / h_{f,y}$		
$F_{c,fc,Rd,y} = 910.17$ [kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]
$W_{pl,z} = 430.18$ [cm <sup>3</sup> ]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]
$M_{c,Rd,z} = 101.09$ [kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
$h_{f,z} = 157$ [mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]
$F_{c,fc,Rd,z} = M_{c,Rd,z} / h_{f,z}$		
$F_{c,fc,Rd,z} = 643.27$ [kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]

### RESISTANCES OF SPREAD FOOTING IN THE COMPRESSION ZONE

$N_{j,Rd} = F_{c,Rd,n}$		
$N_{j,Rd} = 19513.81$ [kN]	Resistance of a spread footing for axial compression	[6.2.8.2.(1)]
$F_{c,Rd,y} = \min(F_{c,Rd,y}, F_{c,fc,Rd,y})$		
$F_{c,Rd,y} = 910.17$ [kN]	Resistance of spread footing in the compression zone	[6.2.8.3]
$F_{c,Rd,z} = \min(F_{c,Rd,z}, F_{c,fc,Rd,z})$		
$F_{c,Rd,z} = 643.27$ [kN]	Resistance of spread footing in the compression zone	[6.2.8.3]

## TENSION ZONE

### STEEL FAILURE

$A_b = 2.45$ [cm <sup>2</sup> ]	Effective anchor area	[Table 3.4]
$f_{ub} = 1000000.00$ [kPa]	Tensile strength of the anchor material	[Table 3.4]
Beta = 0.85	Reduction factor of anchor resistance	[3.6.1.(3)]
$F_{t,Rd,s1} = \beta * 0.9 * f_{ub} * A_b / \gamma_{M2}$		
$F_{t,Rd,s1} = 149.94$ [kN]	Anchor resistance to steel failure	[Table 3.4]
$\gamma_{Ms} = 1.20$	Partial safety factor	CEB [3.2.3.2]
$f_{yb} = 900000.00$ [kPa]	Yield strength of the anchor material	CEB [9.2.2]
$F_{t,Rd,s2} = f_{yb} * A_b / \gamma_{Ms}$		
$F_{t,Rd,s2} = 183.75$ [kN]	Anchor resistance to steel failure	CEB [9.2.2]
$F_{t,Rd,s} = \min(F_{t,Rd,s1}, F_{t,Rd,s2})$		
$F_{t,Rd,s} = 149.94$ [kN]	Anchor resistance to steel failure	

### PULL-OUT FAILURE

$f_{ck} = 382500.00$ [kPa]	Characteristic compressive strength of concrete	EN 1992-1:[3.1.2]
$A_h = 96.86$ [cm <sup>2</sup> ]	Bearing area of the head	CEB [15.1.2.3]
$p_k = 2677500.00$ [kPa]	Characteristic strength of concrete (pull-out)	CEB [15.1.2.3]
$\gamma_{Mp} = 2.16$	Partial safety factor	CEB [3.2.3.1]
$F_{t,Rd,p} = p_k * A_h / \gamma_{Mp}$		
$F_{t,Rd,p} = 12864.01$ [kN]	Design uplift capacity	CEB [9.2.3]

### CONCRETE CONE FAILURE

$h_{ef} = 67$ [mm]	Effective anchorage depth	CEB [9.2.4]
$N_{Rk,c^0} = 7.5 [N^{0.5}/mm^{0.5}] * f_{ck} * h_{ef}^{1.5}$		
$N_{Rk,c^0} = 79.84$ [kN]	Characteristic resistance of an anchor	CEB [9.2.4]
$s_{cr,N} = 200$ [mm]	Critical width of the concrete cone	CEB [9.2.4]
$c_{cr,N} = 100$ [mm]	Critical edge distance	CEB [9.2.4]
$A_{c,N0} = 2500.00$ [cm <sup>2</sup> ]	Maximum area of concrete cone	CEB [9.2.4]
$A_{c,N} = 2500.00$ [cm <sup>2</sup> ]	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]
$c = 100$ [mm]	Minimum edge distance from an anchor	CEB [9.2.4]
$\psi_{s,N} = 0.7 + 0.3 * c / c_{cr,N} \leq 1.0$		

$\psi_{s,N}$	$\frac{1.0}{0}$	Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete	CEB [9.2.4]
$\psi_{ec,N}$	$\frac{1.0}{0}$	Factor related to distribution of tensile forces acting on anchors	CEB [9.2.4]
$\psi_{re,N}$	$0.5 + h_{ef}[mm]/200 \leq 1.0$		
$\psi_{re,N}$	$\frac{1.0}{0}$	Shell spalling factor	CEB [9.2.4]
$\psi_{ucr,N}$	$\frac{1.0}{0}$	Factor taking into account whether the anchorage is in cracked or non-cracked concrete	CEB [9.2.4]
$\gamma_{Mc}$	$\frac{2.1}{6}$	Partial safety factor	CEB [3.2.3.1]
$F_{t,Rd,c}$	$= N_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{ucr,N} / \gamma_{Mc}$		
$F_{t,Rd,c}$	$= 36.96$ [kN]	Design anchor resistance to concrete cone failure	EN 1992-1:[8.4.2.(2)]

### SPLITTING FAILURE

$h_{ef}$	$= 640$ [mm]	Effective anchorage depth	CEB [9.2.5]
$N_{Rk,c}^0$	$= 7.5[N^{0.5}/mm^{0.5}] \cdot f_{ck} \cdot h_{ef}^{1.5}$		
$N_{Rk,c}^0$	$= 2374.91$ [kN]	Design uplift capacity	CEB [9.2.5]
$s_{cr,N}$	$= 1280$ [mm]	Critical width of the concrete cone	CEB [9.2.5]
$c_{cr,N}$	$= 640$ [mm]	Critical edge distance	CEB [9.2.5]
$A_{c,N0}$	$= 24964.00$ [cm <sup>2</sup> ]	Maximum area of concrete cone	CEB [9.2.5]
$A_{c,N}$	$= 2500.00$ [cm <sup>2</sup> ]	Actual area of concrete cone	CEB [9.2.5]
$\psi_{A,N}$	$= A_{c,N} / A_{c,N0}$		
$\psi_{A,N}$	$= 0.10$	Factor related to anchor spacing and edge distance	CEB [9.2.5]
$c$	$= 100$ [mm]	Minimum edge distance from an anchor	CEB [9.2.5]
$\psi_{s,N}$	$= 0.7 + 0.3 \cdot c / c_{cr,N} \leq 1.0$		
$\psi_{s,N}$	$= \frac{0.7}{5}$	Factor taking account the influence of edges of the concrete member on the distribution of stresses in the concrete	CEB [9.2.5]
$\psi_{ec,N}$	$\frac{1.0}{0}$	Factor related to distribution of tensile forces acting on anchors	CEB [9.2.5]
$\psi_{re,N}$	$= 0.5 + h_{ef}[mm]/200 \leq 1.0$		
$\psi_{re,N}$	$\frac{1.0}{0}$	Shell spalling factor	CEB [9.2.5]
$\psi_{ucr,N}$	$\frac{1.0}{0}$	Factor taking into account whether the anchorage is in cracked or non-cracked concrete	CEB [9.2.5]
$\psi_{h,N}$	$= (h / (2 \cdot h_{ef}))^{2/3} \leq 1.2$		
$\psi_{h,N}$	$= 0.79$	Coeff. related to the foundation height	CEB [9.2.5]
$\gamma_{M,sp}$	$= 2.16$	Partial safety factor	CEB [3.2.3.1]
$F_{t,Rd,sp}$	$= N_{Rk,c}^0 \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{ucr,N} \cdot \psi_{h,N} / \gamma_{M,sp}$		
$F_{t,Rd,sp}$	$= 65.03$ [kN]	Design anchor resistance to splitting of concrete	CEB [9.2.5]

### TENSILE RESISTANCE OF AN ANCHOR

$F_{t,Rd}$	$= \min(F_{t,Rd,s}, F_{t,Rd,p}, F_{t,Rd,c}, F_{t,Rd,sp})$		
$F_{t,Rd}$	$= 36.96$ [kN]	Tensile resistance of an anchor	

### BENDING OF THE BASE PLATE

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

$l_{eff,1}$	$= 114$ [mm]	Effective length for a single bolt for mode 1	[6.2.6.5]
$l_{eff,2}$	$= 149$ [mm]	Effective length for a single bolt for mode 2	[6.2.6.5]
$m$	$= 18$ [mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$M_{pl,1,Rd}$	$= 10.76$ [kN*m]	Plastic resistance of a plate for mode 1	[6.2.4]
$M_{pl,2,Rd}$	$= 14.00$ [kN*m]	Plastic resistance of a plate for mode 2	[6.2.4]
$F_{T,1,Rd}$	$= 2362.48$ [kN]	Resistance of a plate for mode 1	[6.2.4]
$F_{T,2,Rd}$	$= 607.14$ [kN]	Resistance of a plate for mode 2	[6.2.4]
$F_{T,3,Rd}$	$= 147.86$ [kN]	Resistance of a plate for mode 3	[6.2.4]
$F_{t,pl,Rd,y}$	$= \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$		
$F_{t,pl,Rd,y}$	$= 147.86$ [kN]	Tension resistance of a plate	[6.2.4]

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

$l_{eff,1} =$	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]
$m =$	139	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$M_{pl,1,Rd} =$	23.50	[kN*m]	Plastic resistance of a plate for mode 1	[6.2.4]
$M_{pl,2,Rd} =$	23.50	[kN*m]	Plastic resistance of a plate for mode 2	[6.2.4]
$F_{T,1,Rd} =$	674.02	[kN]	Resistance of a plate for mode 1	[6.2.4]
$F_{T,2,Rd} =$	258.02	[kN]	Resistance of a plate for mode 2	[6.2.4]
$F_{T,3,Rd} =$	147.86	[kN]	Resistance of a plate for mode 3	[6.2.4]
$F_{t,pl,Rd,z} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$				
$F_{t,pl,Rd,z} =$	147.86	[kN]	Tension resistance of a plate	[6.2.4]

## TENSILE RESISTANCE OF A COLUMN WEB

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

$t_{wc} =$	8	[mm]	Effective thickness of the column web	[6.2.6.3.(8)]
$b_{eff,t,wc} =$	250	[mm]	Effective width of the web for tension	[6.2.6.3.(2)]
$A_{vc} =$	28.76	[cm <sup>2</sup> ]	Shear area	EN1993-1-1:[6.2.6.(3)]
$\omega =$	0.80		Reduction factor for interaction with shear	[6.2.6.3.(4)]
$F_{t,wc,Rd,z} = \omega b_{eff,t,wc} t_{wc} f_{yc} / \gamma_{M0}$				
$F_{t,wc,Rd,z} =$	353.61	[kN]	Column web resistance	[6.2.6.3.(1)]

## RESISTANCES OF SPREAD FOOTING IN THE TENSION ZONE

$F_{T,Rd,y} = F_{t,pl,Rd,y}$				
$F_{T,Rd,y} =$	147.86	[kN]	Resistance of a column base in the tension zone	[6.2.8.3]
$F_{T,Rd,z} = \min(F_{t,pl,Rd,z}, F_{t,wc,Rd,z})$				
$F_{T,Rd,z} =$	147.86	[kN]	Resistance of a column base in the tension zone	[6.2.8.3]

## CONNECTION CAPACITY CHECK

$N_{j,Ed} / N_{j,Rd} \leq 1,0$ (6.24)	$0.01 < 1.00$	verified	(0.01)
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$e_y =$	223	[mm]	Axial force eccentricity	[6.2.8.3]
$Z_{c,y} =$	119	[mm]	Lever arm $F_{C,Rd,y}$	[6.2.8.1.(2)]
$Z_{t,y} =$	150	[mm]	Lever arm $F_{T,Rd,y}$	[6.2.8.1.(3)]
$M_{j,Rd,y} =$	84.91	[kN*m]	Connection resistance for bending	[6.2.8.3]

$M_{j,Ed,y} / M_{j,Rd,y} \leq 1,0$ (6.23)	$0.59 < 1.00$	verified	(0.59)
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$e_z =$	40	[mm]	Axial force eccentricity	[6.2.8.3]
$Z_{c,z} =$	79	[mm]	Lever arm $F_{C,Rd,z}$	[6.2.8.1.(2)]
$Z_{t,z} =$	150	[mm]	Lever arm $F_{T,Rd,z}$	[6.2.8.1.(3)]
$M_{j,Rd,z} =$	34.20	[kN*m]	Connection resistance for bending	[6.2.8.3]

$M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0$ (6.23)	$0.26 < 1.00$	verified	(0.26)
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$M_{j,Ed,y} / M_{j,Rd,y} + M_{j,Ed,z} / M_{j,Rd,z} \leq 1,0$	$0.85 < 1.00$	verified	(0.85)
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## SHEAR

### BEARING PRESSURE OF AN ANCHOR BOLT ONTO THE BASE PLATE

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

$\alpha_{d,y} = 1.52$	Coeff. taking account of the bolt position - in the direction of shear	[Table 3.4]
$\alpha_{b,y} = 1.00$	Coeff. for resistance calculation $F_{1,vb,Rd}$	[Table 3.4]
$k_{1,y} = 2.50$	Coeff. taking account of the bolt position - perpendicularly to the direction of shear	[Table 3.4]
$F_{1,vb,Rd,y} = k_{1,y} \cdot \alpha_{b,y} \cdot f_{up} \cdot d \cdot t_p / \gamma_{M2}$		

$F_{1,vb,Rd,y} = 576.00$  [kN] Resistance of an anchor bolt for bearing pressure onto the base plate [6.2.2.(7)]

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

$\alpha_{d,z} = 1.52$  Coeff. taking account of the bolt position - in the direction of shear [Table 3.4]

$\alpha_{b,z} = 1.00$  Coeff. for resistance calculation  $F_{1,vb,Rd}$  [Table 3.4]

$k_{1,z} = 2.50$  Coeff. taking account of the bolt position - perpendicularly to the direction of shear [Table 3.4]

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

$F_{1,vb,Rd,z} = 576.00$  [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)]

$A_{vb} = 3.14$  [cm<sup>2</sup>] Area of bolt section [6.2.2.(7)]

$f_{ub} = 1000000.00$  [kPa] Tensile strength of the anchor material [6.2.2.(7)]

$\gamma_{M2} = 1.25$  Partial safety factor [6.2.2.(7)]

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

$F_{2,vb,Rd} = 62.33$  [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]

$M_{Rk,s} = 1.04$  [kN\*m] Characteristic bending resistance of an anchor CEB [9.3.2.2]

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

$\gamma_{Ms} = 1.20$  Partial safety factor CEB [3.2.3.2]

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

$F_{v,Rd,sm} = 28.96$  [kN] Shear resistance of a bolt - with lever arm CEB [9.3.1]

#### CONCRETE PRY-OUT FAILURE

$N_{Rk,c} = 79.84$  [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]

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

$F_{v,Rd,cp} = 73.93$  [kN] Concrete resistance for pry-out failure CEB [9.3.1]

#### CONCRETE EDGE FAILURE

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

$V_{Rk,c,y} = 332.21$  [kN] Characteristic resistance of an anchor CEB [9.3.4.(a)]

$\Psi_{A,V,y} = 0.67$  Factor related to anchor spacing and edge distance CEB [9.3.4]

$\Psi_{h,V,y} = 1.00$  Factor related to the foundation thickness CEB [9.3.4.(c)]

$\Psi_{s,V,y} = 0.90$  Factor related to the influence of edges parallel to the shear load direction CEB [9.3.4.(d)]

$\Psi_{ec,V,y} = 1.00$  Factor taking account a group effect when different shear loads are acting on the individual anchors in a group CEB [9.3.4.(e)]

$\Psi_{\alpha,V,y} = 1.00$  Factor related to the angle at which the shear load is applied CEB [9.3.4.(f)]

$\Psi_{ucr,V,y} = 1.00$  Factor related to the type of edge reinforcement used CEB [9.3.4.(g)]

$\gamma_{Mc} = 2.16$  Partial safety factor CEB [3.2.3.1]

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

$F_{v,Rd,c,y} = 92.28$  [kN] Concrete resistance for edge failure CEB [9.3.1]

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

$V_{Rk,c,z} = 332.21$  [kN] Characteristic resistance of an anchor CEB [9.3.4.(a)]

$\Psi_{A,V,z} = 0.67$  Factor related to anchor spacing and edge distance CEB [9.3.4]

$\Psi_{h,V,z} = 1.00$  Factor related to the foundation thickness CEB [9.3.4.(c)]

$V_{Rk,c,z}$	332.0	[kN]	Characteristic resistance of an anchor	CEB [9.3.4.(a)]
$\Psi_{s,V,z}$	0.90		Factor related to the influence of edges parallel to the shear load direction	CEB [9.3.4.(d)]
$\Psi_{ec,V,z}$	1.00		Factor taking account a group effect when different shear loads are acting on the individual anchors in a group	CEB [9.3.4.(e)]
$\Psi_{\alpha,V,z}$	1.00		Factor related to the angle at which the shear load is applied	CEB [9.3.4.(f)]
$\Psi_{ucr,V,z}$	1.00		Factor related to the type of edge reinforcement used	CEB [9.3.4.(g)]
$\gamma_{Mc}$	2.16		Partial safety factor	CEB [3.2.3.1]
$F_{v,Rd,c,z}$	$V_{Rk,c,z} \cdot \Psi_{A,V,z} \cdot \Psi_{h,V,z} \cdot \Psi_{s,V,z} \cdot \Psi_{ec,V,z} \cdot \Psi_{\alpha,V,z} \cdot \Psi_{ucr,V,z} / \gamma_{Mc}$			
$F_{v,Rd,c,z}$	92.28	[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)]
$N_{c,Ed}$	224.00	[kN]	Compressive force	[6.2.2.(6)]
$F_{f,Rd}$	$C_{f,d} \cdot N_{c,Ed}$			
$F_{f,Rd}$	67.20	[kN]	Slip resistance	[6.2.2.(6)]

### BEARING PRESSURE OF THE WEDGE ONTO CONCRETE

$F_{v,Rd,wg,y}$	$1.4 \cdot l_w \cdot b_{wy} \cdot f_{ck} / \gamma_c$			
$F_{v,Rd,wg,y}$	3570.00	[kN]	Resistance for bearing pressure of the wedge onto concrete	
$F_{v,Rd,wg,z}$	$1.4 \cdot l_w \cdot b_{wz} \cdot f_{ck} / \gamma_c$			
$F_{v,Rd,wg,z}$	1963.50	[kN]	Resistance for bearing pressure of the wedge onto concrete	

### SHEAR CHECK

$V_{j,Rd,y}$	$n_b \cdot \min(F_{1,vb,Rd,y}, F_{2,vb,Rd,y}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,y}) + F_{v,Rd,wg,y} + F_{f,Rd}$			
$V_{j,Rd,y}$	3984.67	[kN]	Connection resistance for shear	CEB [9.3.1]
$V_{j,Ed,y} / V_{j,Rd,y} \leq 1,0$	0.00	< 1.00	verified	(0.00)
$V_{j,Rd,z}$	$n_b \cdot \min(F_{1,vb,Rd,z}, F_{2,vb,Rd,z}, F_{v,Rd,sm}, F_{v,Rd,cp}, F_{v,Rd,c,z}) + F_{v,Rd,wg,z} + F_{f,Rd}$			
$V_{j,Rd,z}$	2378.17	[kN]	Connection resistance for shear	CEB [9.3.1]
$V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$	0.01	< 1.00	verified	(0.01)
$V_{j,Ed,y} / V_{j,Rd,y} + V_{j,Ed,z} / V_{j,Rd,z} \leq 1,0$	0.01	< 1.00	verified	(0.01)

### WELDS BETWEEN THE COLUMN AND THE BASE PLATE

$\sigma_{\perp}$	84804.16	[kPa]	Normal stress in a weld	[4.5.3.(7)]
$\tau_{\perp}$	84804.16	[kPa]	Perpendicular tangent stress	[4.5.3.(7)]
$\tau_{yII}$	813.01	[kPa]	Tangent stress parallel to $V_{j,Ed,y}$	[4.5.3.(7)]
$\tau_{zII}$	-10370.37	[kPa]	Tangent stress parallel to $V_{j,Ed,z}$	[4.5.3.(7)]
$\beta_W$	0.80		Resistance-dependent coefficient	[4.5.3.(7)]
$\sigma_{\perp} / (0.9 \cdot f_u / \gamma_{M2}) \leq 1.0$ (4.1)	0.33	< 1.00	verified	(0.33)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{yII}^2 + \tau_{zII}^2))} / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)	0.47	< 1.00	verified	(0.47)
$\sqrt{(\sigma_{\perp}^2 + 3.0 (\tau_{zII}^2 + \tau_{\perp}^2))} / (f_u / (\beta_W \cdot \gamma_{M2})) \leq 1.0$ (4.1)	0.30	< 1.00	verified	(0.30)

### CONNECTION STIFFNESS

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

$b_{eff}$	67	[mm]	Effective width of the bearing pressure zone under the flange	[6.2.5.(3)]
$l_{eff}$	314	[mm]	Effective length of the bearing pressure zone under the flange	[6.2.5.(3)]
$k_{13,y}$	$E_c \cdot \sqrt{(b_{eff} \cdot l_{eff})} / (1.275 \cdot E)$			
$k_{13,y}$	114	[mm]	Stiffness coeff. of compressed concrete	[Table 6.11]

$l_{eff} =$	114	[mm]	Effective length for a single bolt for mode 1	[6.2.6.5]
$m =$	18	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$k_{15,y} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$				
$k_{15,y} =$	515	[mm]	Stiffness coeff. of the base plate subjected to tension	[Table 6.11]
$L_b =$	250	[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]
$\lambda_{0,y} =$	0.49		Column slenderness	[5.2.2.5.(2)]
$S_{j,ini,y} =$	48290.63	[kN*m]	Initial rotational stiffness	[Table 6.12]
$S_{j,rig,y} =$	131733.00	[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)]
<b>Bending moment <math>M_{j,E_d,z}</math></b>				
$k_{13,z} = E_c \cdot \sqrt{(A_{c,z})} / (1.275 \cdot E)$				
$k_{13,z} =$	114	[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]
$m =$	139	[mm]	Distance of a bolt from the stiffening edge	[6.2.6.5]
$k_{15,z} = 0.850 \cdot l_{eff} \cdot t_p^3 / (m^3)$				
$k_{15,z} =$	5	[mm]	Stiffness coeff. of the base plate subjected to tension	[Table 6.11]
$L_b =$	250	[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]
$\lambda_{0,z} =$	0.82		Column slenderness	[5.2.2.5.(2)]
$S_{j,ini,z} =$	294714.16	[kN*m]	Initial rotational stiffness	[6.3.1.(4)]
$S_{j,rig,z} =$	46211.26	[kN*m]	Stiffness of a rigid connection	[5.2.2.5]
$S_{j,ini,z} \geq S_{j,rig,z}$ RIGID				[5.2.2.5.(2)]

**WEAKEST COMPONENT:**

FOUNDATION - CONCRETE CONE PULL-OUT FAILURE

<b>Connection conforms to the code</b>	Ratio	0.85
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