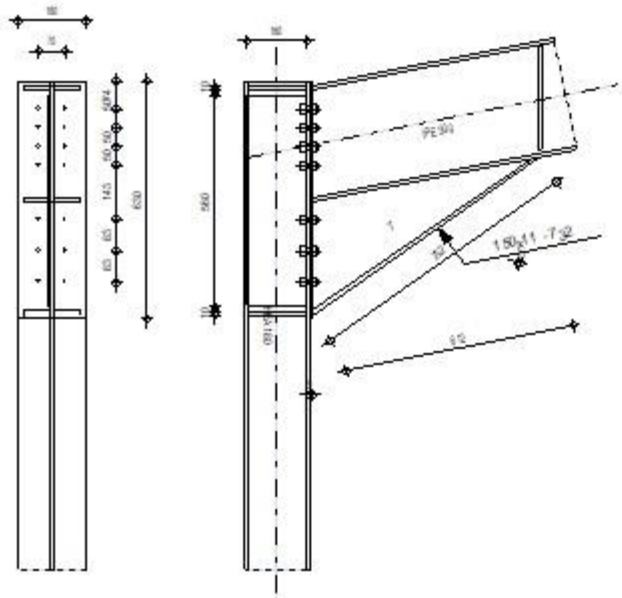
Autodesk Robot Structural Analysis Professional 2020

Design of fixed beam-to-column connection

NF EN 1993-1-8:2005/NA:2007/AC:2009

OK

Ratio
0.59



GENERAL

Connection no.: 7
Connection name: Frame knee
Structure node: 38
Structure bars: 39, 42

GEOMETRY

COLUMN

Section: HEA 180
Bar no.: 39

α =	-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^2]	Cross-sectional area of a column
I_{xc} =	2510.29	[cm^4]	Moment of inertia of the column section

Material: ACIER

f_{yc} =	235.00	[MPa]	Resistance

BEAM

Section:	IPE 300		
Bar no.:	42		
α =	11.3	[Deg]	Inclination angle
h_b =	300	[mm]	Height of beam section
b_f =	150	[mm]	Width of beam section
t_{wb} =	7	[mm]	Thickness of the web of beam section
t_{fb} =	11	[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 =	53.81	[cm^2]	Cross-sectional area of a beam
I_{xb} =	8356.11	[cm^4]	Moment of inertia of the beam section
Material:	ACIER		
f_{yb} =	235.00	[MPa]	Resistance

BOLTS

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

d =	12	[mm]	Bolt diameter
Class =	8.8		Bolt class
F_{tRd} =	48.38	[kN]	Tensile resistance of a bolt
n_h =	2		Number of bolt columns
n_v =	7		Number of bolt rows
h_1 =	74	[mm]	Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 70 [mm]

Vertical spacing p_i = 50;50;50;143;83;83 [mm]

PLATE

h_p =	630	[mm]	Plate height
b_p =	180	[mm]	Plate width
t_p =	9	[mm]	Plate thickness
Material:	ACIER		
f_{yp} =	235.00	[MPa]	Resistance

LOWER STIFFENER

w_d =	150	[mm]	Plate width
t_{fd} =	11	[mm]	Flange thickness
h_d =	300	[mm]	Plate height
t_{wd} =	7	[mm]	Web thickness
l_d =	612	[mm]	Plate length
α =	35.0	[Deg]	Inclination angle
Material:	ACIER		
f_{ybu} =	235.00	[MPa]	Resistance

COLUMN STIFFENER

Upper

h_{su} =	152	[mm]	Stiffener height
b_{su} =	87	[mm]	Stiffener width
t_{hu} =	10	[mm]	Stiffener thickness
Material:	ACIER E24		
f_{ysu} =	235.00	[MPa]	Resistance

Lower

h_{sd} =	152	[mm]	Stiffener height	
b_{sd} =	87	[mm]	Stiffener width	
t_{hd} =	10	[mm]	Stiffener thickness	
Material:	ACIER E24			
f_{ysu} =	235.00	[MPa]	Resistance	

PLATE STRENGTHENING COLUMN WEB

Typ:	unilateral			
h_a =	560	[mm]	Plate length	
w_a =	160	[mm]	Plate width	
t_a =	10	[mm]	Plate thickness	
Material:	ACIER E24			
f_{ya} =	235.00	[MPa]	Resistance	

FILLET WELDS

a_w =	5	[mm]	Web weld	
a_f =	5	[mm]	Flange weld	
a_s =	5	[mm]	Stiffener weld	
a_{fd} =	5	[mm]	Horizontal weld	
a_{p1} =	1	[mm]	Horizontal weld	
a_{p2} =	1	[mm]	Vertical weld	

MATERIAL FACTORS

γ_{M0} =	1.00		Partial safety factor	[2.2]
γ_{M1} =	1.00		Partial safety factor	[2.2]
γ_{M2} =	1.25		Partial safety factor	[2.2]
γ_{M3} =	1.10		Partial safety factor	[2.2]

LOADS

Ultimate limit state

Case: 16: ULS /106/ 1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 5*1.35 + 6*1.35 + 7*1.05 + 8*1.05 + 9*1.05 + 15*1.50

$M_{b1,Ed}$ =	58.22	[kN*m]	Bending moment in the right beam	
$V_{b1,Ed}$ =	42.31	[kN]	Shear force in the right beam	
$N_{b1,Ed}$ =	-22.17	[kN]	Axial force in the right beam	
$M_{c1,Ed}$ =	58.22	[kN*m]	Bending moment in the lower column	
$V_{c1,Ed}$ =	-22.18	[kN]	Shear force in the lower column	
$N_{c1,Ed}$ =	-43.15	[kN]	Axial force in the lower column	

RESULTS

BEAM RESISTANCES

COMPRESSION

A_b =	53.81	[cm^2]	Area	EN1993-1-1:[6.2.4]

$N_{cb,Rd} = A_b f_{yb} / \gamma_{M0}$

$N_{cb,Rd}$ =	1264.54	[kN]	Design compressive resistance of the section	EN1993-1-1:[6.2.4]

SHEAR

A_{vb} =	46.98	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd}$ = 637.41 [kN] Design sectional resistance for shear EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \le 1,0$ 0.07 < 1.00 verified (0.07)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} = 628.36 [cm^3] Plastic section modulus EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb} f_{yb} / \gamma_{M0}$

$M_{b,pl,Rd}$ = 147.66 [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} = 1445.52 [cm^3] Plastic section modulus EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{pl} f_{yb} / \gamma_{M0}$

$M_{cb,Rd}$ = 339.70 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ = 339.70 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

h_f = 594 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd}$ = 571.92 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

WEB OR BRACKET FLANGE - COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

β = 11.3 [Deg] Angle between the front plate and the beam

γ = 35.0 [Deg] Inclination angle of the bracket plate

$b_{eff,c,wb}$ = 169 [mm] Effective width of the web for compression [6.2.6.2.(1)]

A_{vb} = 25.68 [cm^2] Shear area EN1993-1-1:[6.2.6.(3)]

ω = 0.88 Reduction factor for interaction with shear [6.2.6.2.(1)]

$\sigma_{com,Ed}$ = 90.73 [MPa] 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 = 14.29 [cm^2] Area of the web stiffener EN1993-1-1:[6.2.4]

$F_{c,wb,Rd1} = [\omega\ k_{wc}\ b_{eff,c,wb}\ t_{wb}\ f_{yb} / \gamma_{M0} + A_s\ f_{yb} / \gamma_{M0}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd1}$ = 1193.17 [kN] Beam web resistance [6.2.6.2.(1)]

Buckling:

d_{wb} = 249 [mm] Height of compressed web [6.2.6.2.(1)]

λ_p = 0.90 Plate slenderness of an element [6.2.6.2.(1)]

ρ = 0.86 Reduction factor for element buckling [6.2.6.2.(1)]

λ_s = 5.60 Stiffener slenderness EN1993-1-1:[6.3.1.2]

χ = 1.00 Buckling coefficient of the stiffener EN1993-1-1:[6.3.1.2]

$F_{c,wb,Rd2} = [\omega\ k_{wc}\ \rho\ b_{eff,c,wb}\ t_{wb}\ f_{yb} / \gamma_{M1} + A_s\ \chi\ f_{yb} / \gamma_{M1}] \cos(\gamma) / \sin(\gamma - \beta)$

$F_{c,wb,Rd2}$ = 1123.87 [kN] Beam web resistance [6.2.6.2.(1)]

Resistance of the bracket flange

$F_{c,wb,Rd3} = b_b\ t_b\ f_{yb} / (0.8*\gamma_{M0})$

$F_{c,wb,Rd3}$ = 471.47 [kN] Resistance of the bracket flange [6.2.6.7.(1)]

Final resistance:

$F_{c,wb,Rd,low} = \mathrm{Min}\ (F_{c,wb,Rd1}\ ,\ F_{c,wb,Rd2}\ ,\ F_{c,wb,Rd3})$

$F_{c,wb,Rd,low}$ = 471.47 [kN] Beam web resistance [6.2.6.2.(1)]

COLUMN RESISTANCES

WEB PANEL - SHEAR

$M_{b1,Ed}$ = 58.22 [kN*m] Bending moment (right beam) [5.3.(3)]

$M_{b2,Ed}$ = 0.00 [kN*m] Bending moment (left beam) [5.3.(3)]

$V_{c1,Ed}$ = -22.18 [kN] Shear force (lower column) [5.3.(3)]

$V_{c2,Ed}$ = 0.00 [kN] Shear force (upper column) [5.3.(3)]

z = 515 [mm] Lever arm [6.2.5]

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$

$V_{wp,Ed}$ = 124.06 [kN] Shear force acting on the web panel [5.3.(3)]

A_{vs} =	14.47	[cm^2]	Shear area of the column web	EN1993-1-1:[6.2.6.(3)]
A_{vp} =	9.60	[cm^2]	Area of the web stiffening plate	EN1993-1-1:[6.2.6.(3)]
A_{vc} =	24.07	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
d_s =	590	[mm]	Distance between the centroids of stiffeners	[6.2.6.1.(4)]
$M_{pl,fc,Rd}$ =	0.95	[kN*m]	Plastic resistance of the column flange for bending	[6.2.6.1.(4)]
$M_{pl,stu,Rd}$ =	1.06	[kN*m]	Plastic resistance of the upper transverse stiffener for bending	[6.2.6.1.(4)]
$M_{pl,stl,Rd}$ =	1.06	[kN*m]	Plastic resistance of the lower transverse stiffener for bending	[6.2.6.1.(4)]

$V_{wp,Rd} = 0.9\ (A_{vs}*f_{y,wc}+A_{vp}*f_{ya}) / (\sqrt{3}\ \gamma_{M0}) + \mathrm{Min}(4\ M_{pl,fc,Rd} / d_s\ , (2\ M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,stl,Rd}) / d_s)$

$V_{wp,Rd}$ =	300.39	[kN]	Resistance of the column web panel for shear	[6.2.6.1]
$V_{wp,Ed} / V_{wp,Rd} \le 1,0$	0.41 < 1.00		verified	(0.41)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

t_{wc} =	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	168	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	24.07	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0.81		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	151.02	[MPa]	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 =	17.40	[cm^2]	Area of the web stiffener	EN1993-1-1:[6.2.4]

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

$F_{c,wc,Rd1}$ =	697.43	[kN]	Column web resistance	[6.2.6.2.(1)]

Buckling:

d_{wc} =	122	[mm]	Height of compressed web	[6.2.6.2.(1)]
λ_p =	0.50		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	1.00		Reduction factor for element buckling	[6.2.6.2.(1)]
λ_s =	2.31		Stiffener slenderness	EN1993-1-1:[6.3.1.2]
χ_s =	1.00		Buckling coefficient of the stiffener	EN1993-1-1:[6.3.1.2]

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

$F_{c,wc,Rd2}$ =	697.43	[kN]	Column web resistance	[6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,low} = \mathrm{Min}\ (F_{c,wc,Rd1}\ , F_{c,wc,Rd2})$

$F_{c,wc,Rd}$ =	697.43	[kN]	Column web resistance	[6.2.6.2.(1)]

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM TOP FLANGE

Bearing:

t_{wc} =	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	166	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	24.07	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
ω =	0.82		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	151.02	[MPa]	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 =	17.40	[cm^2]	Area of the web stiffener	EN1993-1-1:[6.2.4]

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

$F_{c,wc,Rd1}$ =	694.97	[kN]	Column web resistance	[6.2.6.2.(1)]

Buckling:

d_{wc} =	122	[mm]	Height of compressed web	[6.2.6.2.(1)]
λ_p =	0.49		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	1.00		Reduction factor for element buckling	[6.2.6.2.(1)]
λ_s =	2.31		Stiffener slenderness	EN1993-1-1:[6.3.1.2]
χ_s =	1.00		Buckling coefficient of the stiffener	EN1993-1-1:[6.3.1.2]

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

$F_{c,wc,Rd2}$ =	694.97	[kN]	Column web resistance	[6.2.6.2.(1)]

Final resistance:

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

$F_{c,wc,Rd,upp}$ = 694.97 [kN] Column web resistance [6.2.6.2.(1)]

GEOMETRICAL PARAMETERS OF A CONNECTION

EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	20	-	55	-	50	126	148	126	148	113	99	99	99
2	20	-	55	-	50	126	149	126	149	100	50	50	50
3	20	-	55	-	50	126	149	126	149	100	50	50	50
4	20	-	55	-	97	126	149	126	149	193	97	97	97
5	20	-	55	-	113	126	149	126	149	226	113	113	113
6	20	-	55	-	83	126	149	126	149	166	83	83	83
7	20	-	55	-	83	126	148	126	148	146	115	115	115

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	26	-	55	-	50	162	176	162	176	131	115	115	115
2	26	-	55	-	50	162	172	162	172	100	50	50	50
3	26	-	55	-	50	162	172	162	172	100	50	50	50
4	26	-	55	-	97	162	172	162	172	193	97	97	97
5	26	-	55	-	113	162	172	162	172	226	113	113	113
6	26	-	55	-	83	162	172	162	172	166	83	83	83
7	26	-	55	-	83	162	172	162	172	164	127	127	127

m – Bolt distance from the web
m_x – Bolt distance from the beam flange
e – Bolt distance from the outer edge
e_x – Bolt distance from the horizontal outer edge
p – Distance between bolts
$l_{eff,cp}$ – Effective length for a single bolt in the circular failure mode
$l_{eff,nc}$ – Effective length for a single bolt in the non-circular failure mode
$l_{eff,1}$ – Effective length for a single bolt for mode 1
$l_{eff,2}$ – Effective length for a single bolt for mode 2
$l_{eff,cp,g}$ – Effective length for a group of bolts in the circular failure mode
$l_{eff,nc,g}$ – Effective length for a group of bolts in the non-circular failure mode
$l_{eff,1,g}$ – Effective length for a group of bolts for mode 1
$l_{eff,2,g}$ – Effective length for a group of bolts for mode 2

CONNECTION RESISTANCE FOR COMPRESSION

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

$N_{j,Rd}$ = 942.94 [kN] Connection resistance for compression [6.2]

$N_{b1,Ed}$ / $N_{j,Rd}$ ≤ 1,0 0.02 < 1.00 verified (0.02)

CONNECTION RESISTANCE FOR BENDING

$F_{t,Rd}$ = 48.38 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd}$ = 89.17 [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

$F_{t,fc,Rd}$ = 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)]

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

$F_{t1,Rd,comp}$ - Formula	$F_{t1,Rd,comp}$	Component
$F_{t1,Rd} = Min (F_{t1,Rd,comp})$	82.65	Bolt row resistance
$F_{t,fc,Rd(1)} = 88.73$	88.73	Column flange - tension
$F_{t,wc,Rd(1)} = 233.61$	233.61	Column web - tension
$F_{t,ep,Rd(1)} = 82.65$	82.65	Front plate - tension
$F_{t,wb,Rd(1)} = 270.40$	270.40	Beam web - tension
$B_{p,Rd} = 178.33$	178.33	Bolts due to shear punching
$V_{wp,Rd}/\beta = 300.39$	300.39	Web panel - shear
$F_{c,wc,Rd} = 697.43$	697.43	Column web - compression
$F_{c,fb,Rd} = 571.92$	571.92	Beam flange - compression
$F_{c,wb,Rd} = 471.47$	471.47	Beam web - compression

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t2,Rd} = Min (F_{t2,Rd,comp})$	39.27	Bolt row resistance
$F_{t,fc,Rd(2)} = 88.81$	88.81	Column flange - tension
$F_{t,wc,Rd(2)} = 233.61$	233.61	Column web - tension
$F_{t,ep,Rd(2)} = 81.95$	81.95	Front plate - tension
$F_{t,wb,Rd(2)} = 270.40$	270.40	Beam web - tension
$B_{p,Rd} = 178.33$	178.33	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^1 F_{ti,Rd} = 300.39 - 82.65$	217.74	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{tj,Rd} = 697.43 - 82.65$	614.78	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{tj,Rd} = 571.92 - 82.65$	489.27	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^1 F_{tj,Rd} = 471.47 - 82.65$	388.82	Beam web - compression
$F_{t,fc,Rd(2 + 1)} - \sum_1^1 F_{tj,Rd} = 142.64 - 82.65$	59.99	Column flange - tension - group
$F_{t,wc,Rd(2 + 1)} - \sum_1^1 F_{tj,Rd} = 270.90 - 82.65$	188.25	Column web - tension - group
$F_{t,ep,Rd(2 + 1)} - \sum_1^1 F_{tj,Rd} = 121.92 - 82.65$	39.27	Front plate - tension - group
$F_{t,wb,Rd(2 + 1)} - \sum_1^1 F_{tj,Rd} = 275.65 - 82.65$	193.00	Beam web - tension - group

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t3,Rd} = Min (F_{t3,Rd,comp})$	34.53	Bolt row resistance
$F_{t,fc,Rd(3)} = 88.81$	88.81	Column flange - tension
$F_{t,wc,Rd(3)} = 233.61$	233.61	Column web - tension
$F_{t,ep,Rd(3)} = 81.95$	81.95	Front plate - tension
$F_{t,wb,Rd(3)} = 270.40$	270.40	Beam web - tension
$B_{p,Rd} = 178.33$	178.33	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^2 F_{ti,Rd} = 300.39 - 121.92$	178.47	Web panel - shear
$F_{c,wc,Rd} - \sum_1^2 F_{tj,Rd} = 697.43 - 121.92$	575.51	Column web - compression
$F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 571.92 - 121.92$	450.00	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^2 F_{tj,Rd} = 471.47 - 121.92$	349.55	Beam web - compression
$F_{t,fc,Rd(3 + 2)} - \sum_2^2 F_{tj,Rd} = 106.04 - 39.27$	66.77	Column flange - tension - group
$F_{t,wc,Rd(3 + 2)} - \sum_2^2 F_{tj,Rd} = 189.88 - 39.27$	150.61	Column web - tension - group
$F_{t,fc,Rd(3 + 2 + 1)} - \sum_2^1 F_{tj,Rd} = 208.18 - 121.92$	86.26	Column flange - tension - group
$F_{t,wc,Rd(3 + 2 + 1)} - \sum_2^1 F_{tj,Rd} = 341.98 - 121.92$	220.05	Column web - tension - group
$F_{t,ep,Rd(3 + 2)} - \sum_2^2 F_{tj,Rd} = 73.80 - 39.27$	34.53	Front plate - tension - group
$F_{t,wb,Rd(3 + 2)} - \sum_2^2 F_{tj,Rd} = 166.85 - 39.27$	127.58	Beam web - tension - group
$F_{t,ep,Rd(3 + 2 + 1)} - \sum_2^1 F_{tj,Rd} = 158.82 - 121.92$	36.90	Front plate - tension - group

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t,wb,Rd(3+2+1)} - \sum_2^1 F_{tj,Rd} = 359.07 - 121.92$	237.15	Beam web - tension - group

RESISTANCE OF THE BOLT ROW NO. 4

$F_{t4,Rd,comp}$ - Formula	$F_{t4,Rd,comp}$	Component
$F_{t4,Rd} = \mathrm{Min}(F_{t4,Rd,comp})$	71.22	Bolt row resistance
$F_{t,fc,Rd(4)} = 88.81$	88.81	Column flange - tension
$F_{t,wc,Rd(4)} = 233.61$	233.61	Column web - tension
$F_{t,ep,Rd(4)} = 81.95$	81.95	Front plate - tension
$F_{t,wb,Rd(4)} = 270.40$	270.40	Beam web - tension
$B_{p,Rd} = 178.33$	178.33	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^3 F_{ti,Rd} = 300.39 - 156.45$	143.94	Web panel - shear
$F_{c,wc,Rd} - \sum_1^3 F_{tj,Rd} = 697.43 - 156.45$	540.98	Column web - compression
$F_{c,fb,Rd} - \sum_1^3 F_{tj,Rd} = 571.92 - 156.45$	415.47	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^3 F_{tj,Rd} = 471.47 - 156.45$	315.02	Beam web - compression
$F_{t,fc,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 142.04 - 34.53$	107.51	Column flange - tension - group
$F_{t,wc,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 266.97 - 34.53$	232.44	Column web - tension - group
$F_{t,fc,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 207.59 - 73.80$	133.79	Column flange - tension - group
$F_{t,wc,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 338.66 - 73.80$	264.86	Column web - tension - group
$F_{t,fc,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 284.68 - 156.45$	128.23	Column flange - tension - group
$F_{t,wc,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 446.71 - 156.45$	290.26	Column web - tension - group
$F_{t,ep,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 108.12 - 34.53$	73.59	Front plate - tension - group
$F_{t,wb,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 244.44 - 34.53$	209.91	Beam web - tension - group
$F_{t,ep,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 145.01 - 73.80$	71.22	Front plate - tension - group
$F_{t,wb,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 327.86 - 73.80$	254.06	Beam web - tension - group
$F_{t,ep,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 230.04 - 156.45$	73.59	Front plate - tension - group
$F_{t,wb,Rd(4+3+2+1)} - \sum_3^1 F_{tj,Rd} = 520.08 - 156.45$	363.63	Beam web - tension - group

Additional reduction of the bolt row resistance

$F_{t4,Rd} = F_{t1,Rd}\, h_4/h_1$

$F_{t4,Rd} =$ 59.71 [kN] Reduced bolt row resistance [6.2.7.2.(9)]

$F_{t4,Rd} = F_{t2,Rd}\, h_4/h_2$

$F_{t4,Rd} =$ 31.26 [kN] Reduced bolt row resistance [6.2.7.2.(9)]FRA

RESISTANCE OF THE BOLT ROW NO. 5

$F_{t5,Rd,comp}$ - Formula	$F_{t5,Rd,comp}$	Component
$F_{t5,Rd} = \mathrm{Min}(F_{t5,Rd,comp})$	81.95	Bolt row resistance
$F_{t,fc,Rd(5)} = 88.81$	88.81	Column flange - tension
$F_{t,wc,Rd(5)} = 233.61$	233.61	Column web - tension
$F_{t,ep,Rd(5)} = 81.95$	81.95	Front plate - tension
$F_{t,wb,Rd(5)} = 270.40$	270.40	Beam web - tension
$B_{p,Rd} = 178.33$	178.33	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^4 F_{ti,Rd} = 300.39 - 187.71$	112.68	Web panel - shear
$F_{c,wc,Rd} - \sum_1^4 F_{tj,Rd} = 697.43 - 187.71$	509.72	Column web - compression
$F_{c,fb,Rd} - \sum_1^4 F_{tj,Rd} = 571.92 - 187.71$	384.21	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^4 F_{tj,Rd} = 471.47 - 187.71$	283.76	Beam web - compression
$F_{t,fc,Rd(5+4)} - \sum_4^4 F_{tj,Rd} = 156.89 - 31.26$	125.63	Column flange - tension - group
$F_{t,wc,Rd(5+4)} - \sum_4^4 F_{tj,Rd} = 355.33 - 31.26$	324.07	Column web - tension - group
$F_{t,fc,Rd(5+4+3)} - \sum_4^3 F_{tj,Rd} = 222.43 - 65.79$	156.64	Column flange - tension - group
$F_{t,wc,Rd(5+4+3)} - \sum_4^3 F_{tj,Rd} = 412.26 - 65.79$	346.46	Column web - tension - group
$F_{t,fc,Rd(5+4+3+2)} - \sum_4^2 F_{tj,Rd} = 287.97 - 105.06$	182.91	Column flange - tension - group
$F_{t,wc,Rd(5+4+3+2)} - \sum_4^2 F_{tj,Rd} = 458.73 - 105.06$	353.67	Column web - tension - group
$F_{t,fc,Rd(5+4+3+2+1)} - \sum_4^1 F_{tj,Rd} = 365.07 - 187.71$	177.36	Column flange - tension - group

$F_{t5,Rd,comp}$ - Formula	$F_{t5,Rd,comp}$	Component
$F_{t,wc,Rd(5+4+3+2+1)} - \sum_4^1 F_{tj,Rd} = 526.27 - 187.71$	338.56	Column web - tension - group
$F_{t,ep,Rd(5+4)} - \sum_4^4 F_{tj,Rd} = 141.88 - 31.26$	110.62	Front plate - tension - group
$F_{t,wb,Rd(5+4)} - \sum_4^4 F_{tj,Rd} = 349.55 - 31.26$	318.29	Beam web - tension - group
$F_{t,ep,Rd(5+4+3)} - \sum_4^3 F_{tj,Rd} = 191.51 - 65.79$	125.72	Front plate - tension - group
$F_{t,wb,Rd(5+4+3)} - \sum_4^3 F_{tj,Rd} = 432.98 - 65.79$	367.18	Beam web - tension - group
$F_{t,ep,Rd(5+4+3+2)} - \sum_4^2 F_{tj,Rd} = 228.41 - 105.06$	123.35	Front plate - tension - group
$F_{t,wb,Rd(5+4+3+2)} - \sum_4^2 F_{tj,Rd} = 516.40 - 105.06$	411.34	Beam web - tension - group
$F_{t,ep,Rd(5+4+3+2+1)} - \sum_4^1 F_{tj,Rd} = 313.43 - 187.71$	125.72	Front plate - tension - group
$F_{t,wb,Rd(5+4+3+2+1)} - \sum_4^1 F_{tj,Rd} = 708.62 - 187.71$	520.91	Beam web - tension - group

Additional reduction of the bolt row resistance

$F_{t5,Rd} = F_{t1,Rd}\ h_5/h_1$

$F_{t5,Rd} =$ 37.84 [kN] Reduced bolt row resistance [6.2.7.2.(9)]

$F_{t5,Rd} = F_{t2,Rd}\ h_5/h_2$

$F_{t5,Rd} =$ 19.81 [kN] Reduced bolt row resistance [6.2.7.2.(9)]FRA

RESISTANCE OF THE BOLT ROW NO. 6

$F_{t6,Rd,comp}$ - Formula	$F_{t6,Rd,comp}$	Component
$F_{t6,Rd} = \mathrm{Min}\ (F_{t6,Rd,comp})$	81.95	Bolt row resistance
$F_{t,fc,Rd(6)} = 88.81$	88.81	Column flange - tension
$F_{t,wc,Rd(6)} = 233.61$	233.61	Column web - tension
$F_{t,ep,Rd(6)} = 81.95$	81.95	Front plate - tension
$F_{t,wb,Rd(6)} = 270.40$	270.40	Beam web - tension
$B_{p,Rd} = 178.33$	178.33	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^5 F_{ti,Rd} = 300.39 - 207.52$	92.87	Web panel - shear
$F_{c,wc,Rd} - \sum_1^5 F_{tj,Rd} = 697.43 - 207.52$	489.91	Column web - compression
$F_{c,fb,Rd} - \sum_1^5 F_{tj,Rd} = 571.92 - 207.52$	364.40	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^5 F_{tj,Rd} = 471.47 - 207.52$	263.95	Beam web - compression
$F_{t,fc,Rd(6+5)} - \sum_5^5 F_{tj,Rd} = 153.71 - 19.81$	133.90	Column flange - tension - group
$F_{t,wc,Rd(6+5)} - \sum_5^5 F_{tj,Rd} = 338.00 - 19.81$	318.19	Column web - tension - group
$F_{t,fc,Rd(6+5+4)} - \sum_5^4 F_{tj,Rd} = 230.21 - 51.07$	179.13	Column flange - tension - group
$F_{t,wc,Rd(6+5+4)} - \sum_5^4 F_{tj,Rd} = 444.00 - 51.07$	392.93	Column web - tension - group
$F_{t,fc,Rd(6+5+4+3)} - \sum_5^3 F_{tj,Rd} = 295.75 - 85.60$	210.15	Column flange - tension - group
$F_{t,wc,Rd(6+5+4+3)} - \sum_5^3 F_{tj,Rd} = 484.46 - 85.60$	398.86	Column web - tension - group
$F_{t,fc,Rd(6+5+4+3+2)} - \sum_5^2 F_{tj,Rd} = 361.29 - 124.87$	236.42	Column flange - tension - group
$F_{t,wc,Rd(6+5+4+3+2)} - \sum_5^2 F_{tj,Rd} = 517.17 - 124.87$	392.30	Column web - tension - group
$F_{t,fc,Rd(6+5+4+3+2+1)} - \sum_5^1 F_{tj,Rd} = 438.39 - 207.52$	230.87	Column flange - tension - group
$F_{t,wc,Rd(6+5+4+3+2+1)} - \sum_5^1 F_{tj,Rd} = 564.77 - 207.52$	357.24	Column web - tension - group
$F_{t,ep,Rd(6+5)} - \sum_5^5 F_{tj,Rd} = 139.66 - 19.81$	119.85	Front plate - tension - group
$F_{t,wb,Rd(6+5)} - \sum_5^5 F_{tj,Rd} = 327.03 - 19.81$	307.21	Beam web - tension - group
$F_{t,ep,Rd(6+5+4)} - \sum_5^4 F_{tj,Rd} = 209.25 - 51.07$	158.18	Front plate - tension - group
$F_{t,wb,Rd(6+5+4)} - \sum_5^4 F_{tj,Rd} = 488.04 - 51.07$	436.96	Beam web - tension - group
$F_{t,ep,Rd(6+5+4+3)} - \sum_5^3 F_{tj,Rd} = 252.76 - 85.60$	167.16	Front plate - tension - group
$F_{t,wb,Rd(6+5+4+3)} - \sum_5^3 F_{tj,Rd} = 571.46 - 85.60$	485.86	Beam web - tension - group
$F_{t,ep,Rd(6+5+4+3+2)} - \sum_5^2 F_{tj,Rd} = 289.66 - 124.87$	164.79	Front plate - tension - group
$F_{t,wb,Rd(6+5+4+3+2)} - \sum_5^2 F_{tj,Rd} = 654.89 - 124.87$	530.01	Beam web - tension - group
$F_{t,ep,Rd(6+5+4+3+2+1)} - \sum_5^1 F_{tj,Rd} = 374.68 - 207.52$	167.16	Front plate - tension - group
$F_{t,wb,Rd(6+5+4+3+2+1)} - \sum_5^1 F_{tj,Rd} = 847.11 - 207.52$	639.59	Beam web - tension - group

Additional reduction of the bolt row resistance

$F_{t6,Rd} = F_{t1,Rd}\ h_6/h_1$

$F_{t6,Rd} =$ 25.15 [kN] Reduced bolt row resistance [6.2.7.2.(9)]

$F_{t6,Rd} = F_{t2,Rd}\ h_6/h_2$

$F_{t6,Rd} =$ 13.17 [kN] Reduced bolt row resistance [6.2.7.2.(9)]FRA

RESISTANCE OF THE BOLT ROW NO. 7

$F_{t7,Rd,comp}$ - Formula	$F_{t7,Rd,comp}$	Component
$F_{t7,Rd} = Min (F_{t7,Rd,comp})$	79.70	Bolt row resistance
$F_{t,fc,Rd(7)} = 88.58$	88.58	Column flange - tension
$F_{t,wc,Rd(7)} = 233.61$	233.61	Column web - tension
$F_{t,ep,Rd(7)} = 81.95$	81.95	Front plate - tension
$F_{t,wb,Rd(7)} = 270.40$	270.40	Beam web - tension
$B_{p,Rd} = 178.33$	178.33	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^6 F_{ti,Rd} = 300.39 - 220.69$	79.70	Web panel - shear
$F_{c,wc,Rd} - \sum_1^6 F_{tj,Rd} = 697.43 - 220.69$	476.74	Column web - compression
$F_{c,fb,Rd} - \sum_1^6 F_{tj,Rd} = 571.92 - 220.69$	351.23	Beam flange - compression
$F_{c,wb,Rd} - \sum_1^6 F_{tj,Rd} = 471.47 - 220.69$	250.78	Beam web - compression
$F_{t,fc,Rd(7+6)} - \sum_6^6 F_{tj,Rd} = 154.16 - 13.17$	140.99	Column flange - tension - group
$F_{t,wc,Rd(7+6)} - \sum_6^6 F_{tj,Rd} = 340.49 - 13.17$	327.33	Column web - tension - group
$F_{t,fc,Rd(7+6+5)} - \sum_6^5 F_{tj,Rd} = 234.55 - 32.98$	201.57	Column flange - tension - group
$F_{t,wc,Rd(7+6+5)} - \sum_6^5 F_{tj,Rd} = 459.90 - 32.98$	426.92	Column web - tension - group
$F_{t,fc,Rd(7+6+5+4)} - \sum_6^4 F_{tj,Rd} = 311.05 - 64.24$	246.81	Column flange - tension - group
$F_{t,wc,Rd(7+6+5+4)} - \sum_6^4 F_{tj,Rd} = 525.65 - 64.24$	461.41	Column web - tension - group
$F_{t,fc,Rd(7+6+5+4+3)} - \sum_6^3 F_{tj,Rd} = 376.59 - 98.77$	277.82	Column flange - tension - group
$F_{t,wc,Rd(7+6+5+4+3)} - \sum_6^3 F_{tj,Rd} = 550.51 - 98.77$	451.75	Column web - tension - group
$F_{t,fc,Rd(7+6+5+4+3+2)} - \sum_6^2 F_{tj,Rd} = 442.13 - 138.04$	304.09	Column flange - tension - group
$F_{t,wc,Rd(7+6+5+4+3+2)} - \sum_6^2 F_{tj,Rd} = 570.75 - 138.04$	432.71	Column web - tension - group
$F_{t,fc,Rd(7+6+5+4+3+2+1)} - \sum_6^1 F_{tj,Rd} = 519.23 - 220.69$	298.54	Column flange - tension - group
$F_{t,wc,Rd(7+6+5+4+3+2+1)} - \sum_6^1 F_{tj,Rd} = 600.76 - 220.69$	380.07	Column web - tension - group
$F_{t,ep,Rd(7+6)} - \sum_6^6 F_{tj,Rd} = 142.04 - 13.17$	128.87	Front plate - tension - group
$F_{t,wb,Rd(7+6)} - \sum_6^6 F_{tj,Rd} = 351.15 - 13.17$	337.99	Beam web - tension - group
$F_{t,ep,Rd(7+6+5)} - \sum_6^5 F_{tj,Rd} = 214.33 - 32.98$	181.35	Front plate - tension - group
$F_{t,wb,Rd(7+6+5)} - \sum_6^5 F_{tj,Rd} = 539.70 - 32.98$	506.72	Beam web - tension - group
$F_{t,ep,Rd(7+6+5+4)} - \sum_6^4 F_{tj,Rd} = 283.91 - 64.24$	219.67	Front plate - tension - group
$F_{t,wb,Rd(7+6+5+4)} - \sum_6^4 F_{tj,Rd} = 700.71 - 64.24$	636.47	Beam web - tension - group
$F_{t,ep,Rd(7+6+5+4+3)} - \sum_6^3 F_{tj,Rd} = 345.87 - 98.77$	247.10	Front plate - tension - group
$F_{t,wb,Rd(7+6+5+4+3)} - \sum_6^3 F_{tj,Rd} = 784.13 - 98.77$	685.36	Beam web - tension - group
$F_{t,ep,Rd(7+6+5+4+3+2)} - \sum_6^2 F_{tj,Rd} = 383.72 - 138.04$	245.69	Front plate - tension - group
$F_{t,wb,Rd(7+6+5+4+3+2)} - \sum_6^2 F_{tj,Rd} = 867.56 - 138.04$	729.52	Beam web - tension - group
$F_{t,ep,Rd(7+6+5+4+3+2+1)} - \sum_6^1 F_{tj,Rd} = 468.75 - 220.69$	248.06	Front plate - tension - group
$F_{t,wb,Rd(7+6+5+4+3+2+1)} - \sum_6^1 F_{tj,Rd} = 1059.78 - 220.69$	839.09	Beam web - tension - group

Additional reduction of the bolt row resistance

$F_{t7,Rd} = F_{t1,Rd}\ h_7/h_1$

$F_{t7,Rd} =$ 12.45 [kN] Reduced bolt row resistance [6.2.7.2.(9)]

$F_{t7,Rd} = F_{t2,Rd}\ h_7/h_2$

$F_{t7,Rd} =$ 6.52 [kN] Reduced bolt row resistance [6.2.7.2.(9)]FRA

SUMMARY TABLE OF FORCES

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
1	540	82.65	88.73	233.61	82.65	270.40	96.77	178.33
2	490	39.27	88.81	233.61	81.95	270.40	96.77	178.33
3	440	34.53	88.81	233.61	81.95	270.40	96.77	178.33
4	390	31.26	88.81	233.61	81.95	270.40	96.77	178.33
5	247	19.81	88.81	233.61	81.95	270.40	96.77	178.33

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
6	164	13.17	88.81	233.61	81.95	270.40	96.77	178.33
7	81	6.52	88.58	233.61	81.95	270.40	96.77	178.33

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

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

$M_{j,Rd}$ =	98.93	[kN*m]	Connection resistance for bending	[6.2]
$M_{b1,Ed} / M_{j,Rd} \leq 1,0$	0.59 < 1.00		verified	(0.59)

CONNECTION RESISTANCE FOR SHEAR

α_v =	0.60		Coefficient for calculation of $F_{v,Rd}$	[Table 3.4]
β_{Lf} =	0.88		Reduction factor for long connections	[3.8]
$F_{v,Rd}$ =	38.38	[kN]	Shear resistance of a single bolt	[Table 3.4]
$F_{t,Rd,max}$ =	48.38	[kN]	Tensile resistance of a single bolt	[Table 3.4]
$F_{b,Rd,int}$ =	78.84	[kN]	Bearing resistance of an intermediate bolt	[Table 3.4]
$F_{b,Rd,ext}$ =	78.84	[kN]	Bearing resistance of an outermost bolt	[Table 3.4]

Nr	$F_{tj,Rd,N}$	$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	$F_{tj,Ed,M}$	$F_{tj,Ed}$	$F_{vj,Rd}$
1	96.77	-3.17	82.65	48.64	45.48	50.99
2	96.77	-3.17	39.27	23.11	19.94	65.46
3	96.77	-3.17	34.53	20.32	17.15	67.04
4	96.77	-3.17	31.26	18.40	15.23	68.13
5	96.77	-3.17	19.81	11.66	8.49	71.95
6	96.77	-3.17	13.17	7.75	4.58	74.17
7	96.77	-3.17	6.52	3.84	0.67	76.38

$F_{tj,Rd,N}$ – Bolt row resistance for simple tension
$F_{tj,Ed,N}$ – Force due to axial force in a bolt row
$F_{tj,Rd,M}$ – Bolt row resistance for simple bending
$F_{tj,Ed,M}$ – Force due to moment in a bolt row
$F_{tj,Ed}$ – Maximum tensile force in a bolt row
$F_{vj,Rd}$ – Reduced bolt row resistance

$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$
$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$
$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$
$F_{vj,Rd} = Min\ (n_h F_{v,Ed} (1 - F_{tj,Ed}/ (1.4\ n_h F_{t,Rd,max}),\ n_h F_{v,Rd}\ ,\ n_h F_{b,Rd}))$

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

$V_{j,Rd}$ =	474.12	[kN]	Connection resistance for shear	[Table 3.4]
$V_{b1,Ed} / V_{j,Rd} \leq 1,0$	0.09 < 1.00		verified	(0.09)

WELD RESISTANCE

A_w =	93.63	[cm^2]	Area of all welds	[4.5.3.2(2)]
A_{wy} =	39.58	[cm^2]	Area of horizontal welds	[4.5.3.2(2)]
A_{wz} =	54.05	[cm^2]	Area of vertical welds	[4.5.3.2(2)]
I_{wy} =	39366.74	[cm^4]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{\perp max}=\tau_{\perp max}$ =	-34.76	[MPa]	Normal stress in a weld	[4.5.3.2(6)]
$\sigma_{\perp}=\tau_{\perp}$ =	-33.03	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
τ_{II} =	7.83	[MPa]	Tangent stress	[4.5.3.2(5)]
β_w =	0.80		Correlation coefficient	[4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	69.52 < 365.00	verified	(0.19)
$\sqrt{[\sigma_{\perp}^2 + 3*(\tau_{\perp}^2+\tau_{II}^2)]} \leq f_u/(\beta_w*\gamma_{M2})$	67.44 < 365.00	verified	(0.18)
$\sigma_{\perp} \leq 0.9*f_u/\gamma_{M2}$	34.76 < 262.80	verified	(0.13)

CONNECTION STIFFNESS

t_{wash} =	3	[mm]	Washer thickness	[6.2.6.3.(2)]
h_{head} =	9	[mm]	Bolt head height	[6.2.6.3.(2)]
h_{nut} =	12	[mm]	Bolt nut height	[6.2.6.3.(2)]
L_b =	35	[mm]	Bolt length	[6.2.6.3.(2)]
k_{10} =	4	[mm]	Stiffness coefficient of bolts	[6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	hj	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j}\ h_j$	$k_{eff,j}\ h_j^2$
					Sum	20.04	807.82
1	540	3	10	4	1	5.64	305.00
2	490	1	5	2	1	2.86	140.26
3	440	1	5	2	1	2.57	113.12
4	390	3	9	4	1	3.85	150.33
5	247	3	11	4	1	2.74	67.72
6	164	2	8	3	1	1.45	23.78
7	81	3	11	5	1	0.93	7.61

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

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

z_{eq} = 403 [mm] Equivalent force arm [6.3.3.1.(3)]

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

k_{eq} = 5 [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.3.1.(1)]

A_{vc} =	24.07	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
β =	1.00		Transformation parameter	[5.3.(7)]
z =	403	[mm]	Lever arm	[6.2.5]
k_1 =	2	[mm]	Stiffness coefficient of the column web panel subjected to shear	[6.3.2.(1)]
k_2 =	∞		Stiffness coefficient of the compressed column web	[6.3.2.(1)]

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

$S_{j,ini}$ = 53160.24 [kN*m] Initial rotational stiffness [6.3.1.(4)]

μ = 1.00 Stiffness coefficient of a connection [6.3.1.(6)]

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

S_j = 53160.24 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig}$ = 22942.75 [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,pin}$ = 1433.92 [kN*m] Stiffness of a pinned connection [5.2.2.5]

$S_{j,ini} \ge S_{j,rig}$ RIGID

WEAKEST COMPONENT:

FRONT PLATE - TENSION

REMARKS

Distance of bolts from an edge is too large. 97 [mm] > 76 [mm]

Bolts vertical spacing is too large. 143 [mm] > 126 [mm]

Connection conforms to the code	Ratio	0.59