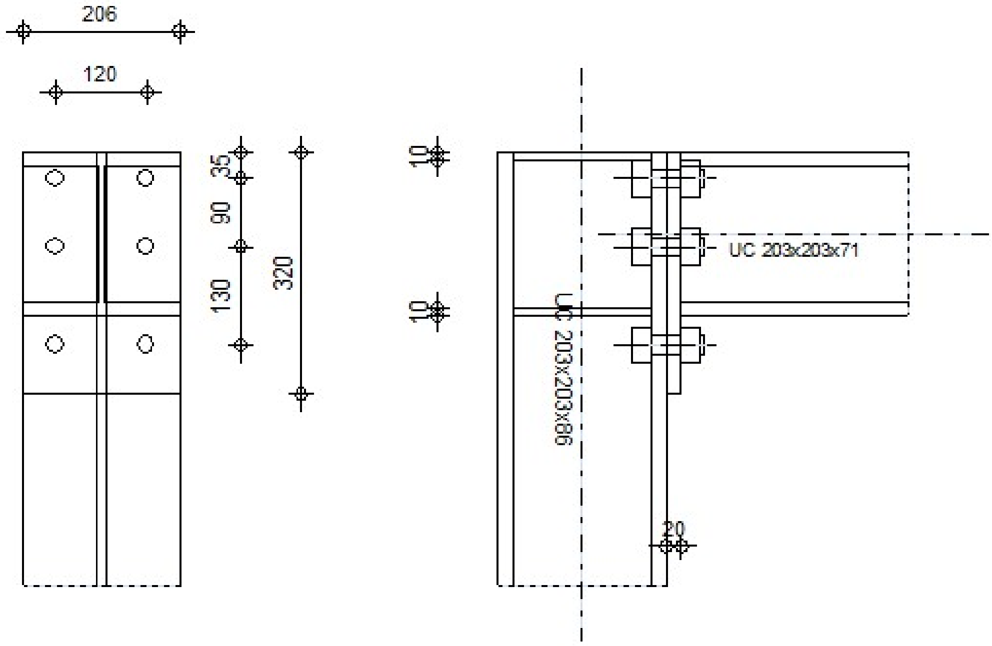
Autodesk Robot Structural Analysis Professional 2014

Design of fixed beam-to-column connection

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

OK

Ratio **0.76**



GENERAL

Connection no.: 19
Connection name: Frame knee

GEOMETRY

COLUMN

Section: UC 203x203x86

α =	-90.0	[Deg]	Inclination angle
h_c =	222	[mm]	Height of column section
b_{fc} =	209	[mm]	Width of column section
t_{wc} =	13	[mm]	Thickness of the web of column section
t_{fc} =	21	[mm]	Thickness of the flange of column section
r_c =	10	[mm]	Radius of column section fillet
A_c =	11000	[mm^2]	Cross-sectional area of a column
I_{xc} =	94490000	[mm^4]	Moment of inertia of the column section

Material: S355

f_{yc} =	355.00	[MPa]	Resistance

BEAM

Section: UC 203x203x71

α =	0.0	[Deg]	Inclination angle

α =	0.0	[Deg]	Inclination angle
h_b =	216	[mm]	Height of beam section
b_f =	206	[mm]	Width of beam section
t_{wb} =	10	[mm]	Thickness of the web of beam section
t_{fb} =	17	[mm]	Thickness of the flange of beam section
r_b =	10	[mm]	Radius of beam section fillet
r_b =	10	[mm]	Radius of beam section fillet
A_b =	9040	[mm^2]	Cross-sectional area of a beam
I_{xb} =	76180000	[mm^4]	Moment of inertia of the beam section
Material:	S355		
f_{yb} =	355.00	[MPa]	Resistance

BOLTS

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

d =	24	[mm]	Bolt diameter
Class =	8.8		Bolt class
F_{tRd} =	203.33	[kN]	Tensile resistance of a bolt
n_h =	2		Number of bolt columns
n_v =	3		Number of bolt rows
h_1 =	35	[mm]	Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 120 [mm]
Vertical spacing p_i = 90;130 [mm]

PLATE

h_p =	320	[mm]	Plate height
b_p =	206	[mm]	Plate width
t_p =	20	[mm]	Plate thickness
Material:	S275		
f_{yp} =	275.00	[MPa]	Resistance

COLUMN STIFFENER

Upper

h_{su} =	181	[mm]	Stiffener height
b_{su} =	98	[mm]	Stiffener width
t_{hu} =	10	[mm]	Stiffener thickness
Material:	S275		
f_{ysu} =	275.00	[MPa]	Resistance

Lower

h_{sd} =	181	[mm]	Stiffener height
b_{sd} =	98	[mm]	Stiffener width
t_{hd} =	10	[mm]	Stiffener thickness
Material:	S275		
f_{ysu} =	275.00	[MPa]	Resistance

FILLET WELDS

a_w =	6	[mm]	Web weld
a_f =	6	[mm]	Flange weld
a_s =	6	[mm]	Stiffener weld

MATERIAL FACTORS

γ_{M0} =	1.00	Partial safety factor	[2.2]

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} =	1.25	Partial safety factor	[2.2]

LOADS

Ultimate limit state

Case: Manual calculations.

$M_{b1,Ed}$ =	58.00	[kN*m]	Bending moment in the right beam
$V_{b1,Ed}$ =	75.00	[kN]	Shear force in the right beam
$N_{b1,Ed}$ =	75.00	[kN]	Axial force in the right beam

RESULTS

BEAM RESISTANCES

TENSION

A_b = 9040 [mm^2] Area EN1993-1-1:[6.2.3]

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

$N_{tb,Rd}$ = 3209.20 [kN] Design tensile resistance of the section EN1993-1-1:[6.2.3]

SHEAR

A_{vb} = 2424 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

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

$V_{cb,Rd}$ = 496.92 [kN] Design sectional resistance for shear EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \leq 1{,}0$ 0.15 < 1.00 verified (0.15)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} = 799000 [mm^3] Plastic section modulus EN1993-1-1:[6.2.5.(2)]

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

$M_{b,pl,Rd}$ = 283.64 [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} = 799000 [mm^3] Plastic section modulus EN1993-1-1:[6.2.5]

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

$M_{cb,Rd}$ = 283.64 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ = 283.64 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

h_f = 198 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

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

$F_{c,fb,Rd}$ = 1428.94 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

COLUMN RESISTANCES

WEB PANEL - SHEAR

$M_{b1,Ed}$ =	58.00	[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)]
$V_{c2,Ed}$ =	0.00	[kN]	Shear force (upper column)	[5.3.(3)]
z =	127	[mm]	Lever arm	[6.2.5]

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

$V_{wp,Ed}$ = 456.15 [kN] Shear force acting on the web panel [5.3.(3)]

A_{vs} = 3105 [mm^2] Shear area of the column web EN1993-1-1:[6.2.6.(3)]

A_{vc} = 3105 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

A_{vs} = 3105 [mm^2] Shear area of the column web EN1993-1-1:[6.2.6.(3)]
d_s = 206 [mm] Distance between the centroids of stiffeners [6.2.6.1.(4)]
$M_{pl,fc,Rd}$ = 7.80 [kN*m] Plastic resistance of the column flange for bending [6.2.6.1.(4)]
$M_{pl,stu,Rd}$ = 1.44 [kN*m] Plastic resistance of the upper transverse stiffener for bending [6.2.6.1.(4)]
$M_{pl,stl,Rd}$ = 1.44 [kN*m] Plastic resistance of the lower transverse stiffener for bending [6.2.6.1.(4)]
$V_{wp,Rd} = 0.9 (A_{vs}*f_{y,wc}) / (\sqrt{3} \gamma_{M0}) + Min(4 M_{pl,fc,Rd} / d_s , (2 M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,stl,Rd}) / d_s)$
$V_{wp,Rd}$ = 662.60 [kN] Resistance of the column web panel for shear [6.2.6.1]
$V_{wp,Ed} / V_{wp,Rd} \leq 1,0$ 0.69 < 1.00 verified (0.69)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:
t_{wc} = 13 [mm] Effective thickness of the column web [6.2.6.2.(6)]
$b_{eff,c,wc}$ = 228 [mm] Effective width of the web for compression [6.2.6.2.(1)]
A_{vc} = 3105 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]
ω = 0.69 Reduction factor for interaction with shear [6.2.6.2.(1)]
$\sigma_{com,Ed}$ = 0.00 [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 = 1964 [mm^2] Area of the web stiffener EN1993-1-1:[6.2.4]
$F_{c,wc,Rd1} = \omega k_{wc} b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M0} + A_s f_{ys} / \gamma_{M0}$
$F_{c,wc,Rd1}$ = 1244.06 [kN] Column web resistance [6.2.6.2.(1)]

Buckling:
d_{wc} = 161 [mm] Height of compressed web [6.2.6.2.(1)]
λ_p = 0.58 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 = 2.58 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]
$F_{c,wc,Rd2} = \omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{yc} / \gamma_{M1} + A_s \chi_s f_{ys} / \gamma_{M1}$
$F_{c,wc,Rd2}$ = 1244.06 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:
$F_{c,wc,Rd,low} = Min (F_{c,wc,Rd1} , F_{c,wc,Rd2})$
$F_{c,wc,Rd}$ = 1244.06 [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	45	-	45	-	90	286	386	286	386	233	312	233	312
2	45	-	45	-	90	286	245	245	245	233	171	171	171
3	45	-	45	-	96	286	280	280	280	239	209	209	209

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	48	-	43	-	90	303	401	303	401	241	322	241	322
2	48	-	43	-	90	303	247	247	247	241	168	168	168
3	48	32	43	65	96	188	103	103	103	-	-	-	-

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

m – Bolt distance from the web
$l_{eff,cp,g}$ – 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 TENSION

$F_{t,Rd}$ =	203.33	[kN]	Bolt resistance for tension	[Table 3.4]
$B_{p,Rd}$ =	466.87	[kN]	Punching shear resistance of a bolt	[Table 3.4]

$N_{j,Rd} = \text{Min}(N_{tb,Rd}, n_v\, n_h\, F_{t,Rd}, n_v\, n_h\, B_{p,Rd})$

$N_{j,Rd}$ =	1219.97	[kN]	Connection resistance for tension	[6.2]

$N_{b1,Ed} / N_{j,Rd} \leq 1,0$	0.06 < 1.00	verified	(0.06)

CONNECTION RESISTANCE FOR BENDING

$F_{t,Rd}$ =	203.33	[kN]	Bolt resistance for tension	[Table 3.4]
$B_{p,Rd}$ =	466.87	[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

$F_{t1,Rd,comp}$ - Formula	$F_{t1,Rd,comp}$	Component
$F_{t1,Rd} = \text{Min}(F_{t1,Rd,comp})$	406.66	Bolt row resistance
$F_{t,fc,Rd(1)} = 406.66$	406.66	Column flange - tension
$F_{t,wc,Rd(1)} = 773.40$	773.40	Column web - tension
$F_{t,ep,Rd(1)} = 406.66$	406.66	Front plate - tension
$F_{t,wb,Rd(1)} = 1075.38$	1075.38	Beam web - tension
$B_{p,Rd} = 933.73$	933.73	Bolts due to shear punching
$V_{wp,Rd}/\beta = 662.60$	662.60	Web panel - shear
$F_{c,wc,Rd} = 1244.06$	1244.06	Column web - compression
$F_{c,fb,Rd} = 1428.94$	1428.94	Beam flange - compression

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t2,Rd} = \text{Min}(F_{t2,Rd,comp})$	255.95	Bolt row resistance
$F_{t,fc,Rd(2)} = 404.13$	404.13	Column flange - tension
$F_{t,wc,Rd(2)} = 727.52$	727.52	Column web - tension
$F_{t,ep,Rd(2)} = 340.70$	340.70	Front plate - tension
$F_{t,wb,Rd(2)} = 876.31$	876.31	Beam web - tension
$B_{p,Rd} = 933.73$	933.73	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^1 F_{ti,Rd} = 662.60 - 406.66$	255.95	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{tj,Rd} = 1244.06 - 406.66$	837.40	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{tj,Rd} = 1428.94 - 406.66$	1022.29	Beam flange - compression
$F_{t,fc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 802.58 - 406.66$	395.93	Column flange - tension - group
$F_{t,wc,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 878.29 - 406.66$	471.63	Column web - tension - group
$F_{t,ep,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 679.70 - 406.66$	273.04	Front plate - tension - group

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t,wb,Rd(2+1)} - \sum_1^1 F_{tj,Rd} = 1714.38 - 406.66$	1307.72	Beam web - tension - group

Additional reduction of the bolt row resistance

$F_{t2,Rd} = F_{t1,Rd}\, h_2/h_1$

$F_{t2,Rd} =$ 194.06 [kN] Reduced bolt row resistance [6.2.7.2.(9)]

The remaining bolts are inactive (they do not carry loads) because resistance of one of the connection components has been used up or these bolts are positioned below the center of rotation.

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	172	406.66	406.66	773.40	406.66	1075.38	406.66	933.73
2	82	194.06	404.13	727.52	340.70	876.31	406.66	933.73
3	-48	-	406.66	767.78	303.75	-	406.66	933.73

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

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

$M_{j,Rd} =$ 85.95 [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \le 1,0$ 0.67 < 1.00 verified (0.67)

CONNECTION RESISTANCE FOR SHEAR

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

$F_{v,Rd} =$ 173.72 [kN] Shear resistance of a single bolt [Table 3.4]

$F_{t,Rd,max} =$ 203.33 [kN] Tensile resistance of a single bolt [Table 3.4]

$F_{b,Rd,int} =$ 373.11 [kN] Bearing resistance of an intermediate bolt [Table 3.4]

$F_{b,Rd,ext} =$ 185.23 [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	406.66	25.00	406.66	274.42	299.42	164.71
2	406.66	25.00	194.06	130.96	155.96	252.26
3	406.66	25.00	0.00	0.00	25.00	332.18

$F_{tj,Rd,N}$ – 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

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

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

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

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

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

$V_{j,Rd} =$ 749.15 [kN] Connection resistance for shear [Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \le 1,0$ 0.10 < 1.00 verified (0.10)

WELD RESISTANCE

$A_w =$ 5280 [mm^2] Area of all welds [4.5.3.2(2)]

$A_{wy} =$ 3350 [mm^2] Area of horizontal welds [4.5.3.2(2)]

$A_{wz} =$ 1930 [mm^2] Area of vertical welds [4.5.3.2(2)]

$I_{wy} =$ 32033301 [mm^4] Moment of inertia of the weld arrangement with respect to the hor. axis [4.5.3.2(5)]

$\sigma_{\perp max}=\tau_{\perp max} =$ 154.77 [MPa] Normal stress in a weld [4.5.3.2(5)]

$\sigma_{\perp}=\tau_{\perp} =$ 145.86 [MPa] Stress in a vertical weld [4.5.3.2(5)]

$\tau_{II} =$ 38.87 [MPa] Tangent stress [4.5.3.2(5)]

$\beta_w =$ 0.85 Correlation coefficient [4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \le f_u/(\beta_w*\gamma_{M2})$	309.54 < 404.71	verified	(0.76)
$\sqrt{[\sigma_{\perp}^2 + 3*(\tau_{\perp}^2+\tau_{II}^2)]} \le f_u/(\beta_w*\gamma_{M2})$	299.39 < 404.71	verified	(0.74)
$\sigma_{\perp} \le 0.9*f_u/\gamma_{M2}$	154.77 < 309.60	verified	(0.50)

CONNECTION STIFFNESS

t_{wash} =	5	[mm]	Washer thickness	[6.2.6.3.(2)]
h_{head} =	17	[mm]	Bolt head height	[6.2.6.3.(2)]
h_{nut} =	24	[mm]	Bolt nut height	[6.2.6.3.(2)]
L_b =	71	[mm]	Bolt length	[6.2.6.3.(2)]
k_{10} =	8	[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	723	106394
1	172	11	19	16	3	522	89858
2	82	8	14	11	2	201	16535

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

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

z_{eq} = 147 [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} =	3105	[mm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
β =	1.00		Transformation parameter	[5.3.(7)]
z =	127	[mm]	Lever arm	[6.2.5]
k_1 =	9	[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)]

$S_{j,ini}$ = 14257.58 [kN*m] Initial rotational stiffness [6.3.1.(4)]

μ = 1.03 Stiffness coefficient of a connection [6.3.1.(6)]

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

S_j = 13796.67 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig}$ = 78084.50 [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,pin}$ = 1561.69 [kN*m] Stiffness of a pinned connection [5.2.2.5]

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

WEAKEST COMPONENT:

WELDS

Connection conforms to the code — Ratio 0.76