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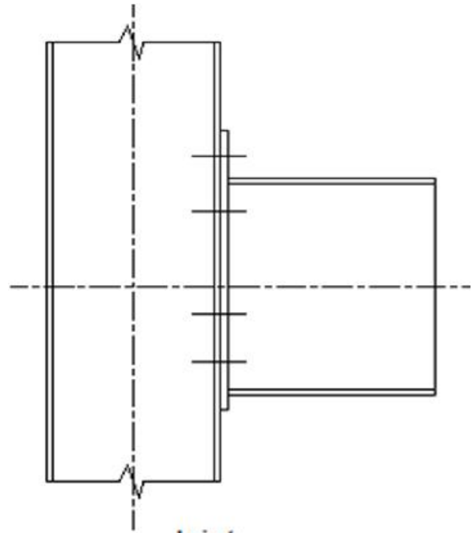
Appendix A:

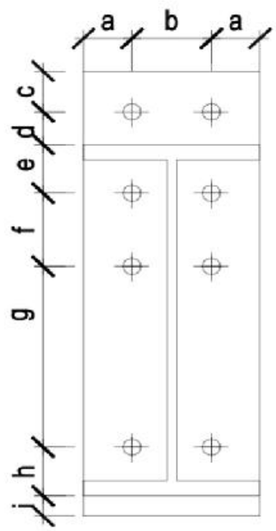
Design Calculations for Extended End Plate Connection- EEP 3d

Reference	Calculation	Output

Connection Detail - EEP 3d



Joint



Row 1	a	=	75	mm
	b	=	100	mm
Row 2	c	=	50	mm
	d	=	40	mm
	e	=	60	mm
Row 3	f	=	90	mm
	g	=	345	mm
	h	=	60	mm
	i	=	25	mm

Beam 533x210x92

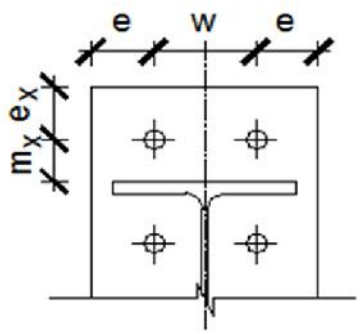
Beam height	=	533.1	mm
Flange width	=	209.3	mm
Mass per 1m length	=	92	kg/m
Flange thickness	=	15.6	mm
Web thickness	=	10.1	mm
Radius of gyration	=	12.7	mm

Column 254x254x107

column height	=	266.7	mm
Flange width	=	258.8	mm
Mass per 1m length	=	107	kg/m
Flange thickness	=	20.5	mm
Web thickness	=	12.8	mm
Radius of gyration	=	12.7	mm

Yield strength	$f_{y,p}$	=	265	N/mm^2
Ultimate tensile strength	$f_{u,p}$	=	410	N/mm^2

Reference	Calculation	Output
	End plate	
	End plate thickness t_p = 25 mm	
	End plate height h_p = 670 mm	
	End plate width w_p = 250 mm	
	Yield strength $f_{y,p}$ = 265 N/mm^2	
	Ultimate tensile strength $f_{u,p}$ = 410 N/mm^2	
	Bolts (class 8.8)	
	Bolt Diameter = 24 mm d_w = 39.6 mm	
	Tensile stress area = 353 mm^2	
	Total no of bolts = 8	
	no of bolts in tension = 6	
	no of bolts in shear = 8	
T 3.1	Yield strength f_{yb} = 640 N/mm^2	
	Ultimate tensile strength f_{ub} = 800 N/mm^2	
	fillet weld thickness	
	Beam flange to end plate weld thickness = 12 mm	
	Beam web to end plate weld thickness = 8 mm	

Reference	Calculation	Output
	Design Calculation according to EC3 **for EEP 3d**	
	Partial factors for Resistance	
	Structural Steel	
EN 1993-1-8	γ_{MO} = 1.0	
N.A.2.15		
T NA.1	γ_{M1} = 1.00 (Resistance of a member to buckling)	
T NA.1	γ_{M2} = 1.10 (plates in bearing in bolted connections)	
	For tring resistance verification $\gamma_{M,u}$ = 1.10	
T NA.1	**Bolts** γ_{M2} = 1.25	
T NA.1	**Welds** γ_{M2} = 1.25	
EN 1993-1-8 :2005	**1. Bolts Tension**	
Cl.3.6.1 (1) T 3.4	$F_{t,Rd} = \dfrac{k_2 f_{ub} A_s}{\gamma_{M2}}$	
	For non countersunk Bolts , k_2 = 0.9	F_1 =
	$F_{t,Rd} = \dfrac{k_2 f_{ub} A_s}{\gamma_{M2}} = \dfrac{0.9 \times 800 \times 353}{1.25}$ = 203.328 kN	203.33 kN
Cl.6.2.6.5	**2. End plate in bending**	
Cl.6.2.6.5 (1)	Bolt row 1 - Bolt row outside tension flange of beam	
T 6.6	Effective length for an end plate	
	for circular patterns, $\ell_{eff,cp}$ = Min ($2\pi m_x$, $\pi m_x + w$, $\pi m_x + 2e$)	
	for extended part of end plate	
	w = 100 mm	
	m_x = 30.40 mm	
	e = 75 mm	
	e_x = 50 mm	
	e_{min} = 75 mm	
	bp = 250 mm	
		
	$2\pi m_x$ = 190.912 mm	
	$\pi m_x + w$ = 195.456 mm	
	$\pi m_x + 2e$ = 245.456 mm	
	$\ell_{eff,cp}$ = 190.91 mm	
	for non circular patterns,	
	$\ell_{eff,nc}$ = Min ($4m_x + 1.25\,e_x$, $e + 2m_x + 0.625\,e_x$, 0.5bp	
	$0.5w + 2m_x + 0.625\,e_x$)	
	$4m_x + 1.25\,e_x$ = 184.1 mm	
	$e + 2m_x + 0.625\,e_x$ = 167.05 mm	
	0.5bp = 125.0 mm	
	$0.5w + 2m_x + 0.625\,e_x$ = 142.05 mm	
	$\ell_{eff,nc}$ = 125.00 mm	
	Mode 1 - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 125.00 mm	

Reference	Calculation	Output
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	$n = e_{min}$ but $n \leq 1.25m$ (38)	
	n = 38.00 mm	
	$M_{p,1,Rd} = \frac{0.25 \Sigma \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 125.00 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 5175.8 kNmm	
	$F_{T,1,Rd}$ = 900 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 125.00 mm	
T 6.2	$F_{T,2,Rd} = \frac{2 M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m + n}$	
	$M_{p,1,Rd} = \frac{0.25 \Sigma \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 125.00 * 25^2 * 265}{1.0}$	
	$M_{p\,2,Rd}$ = 5175.8 kNmm	
	$n = e_{min}$ but $n \leq 1.25m$ (38)	
	n = 38.00 mm	
	$F_{t,Rd} = \frac{0.9 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \times 800 \times 353}{1.25}$ = 203.328 kN	
	$F_{T,2,Rd}$ = 377.26 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 1 bolts = 377.26 kN	
Cl.6.2.6.5 (1)	Bolt row 2 - First Bolt row below tension flange of beam	
T 6.6	Effective length for an end plate,	
	for circular patterns, $\ell_{eff,cp} = 2\pi m$	
	m = 38.55 mm	
	$\ell_{eff,cp}$ = 242.09 mm	
	for non circular patterns, $\ell_{eff,nc} = \alpha m$	
	m_2 = 34.8 mm	
Figure 6.11	$\lambda_1 = \frac{m}{m + e}$ = 0.34 , $\lambda_2 = \frac{m_2}{m + e}$ = 0.31	
	α = 7.5	
	$\ell_{eff,nc}$ = 289.13 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 242.09 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	n = 48.19 mm	

Reference	Calculation	Output
	$M_{p,1,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_p^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 242.09 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 10024.2 kNmm	
	$F_{T,1,Rd}$ = 1283 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 289.13 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	
	$M_{p,2,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_p^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 289.13 * 25^2 * 265}{1.0}$	
	$M_{p\ 2,Rd}$ = 11971.6 kNmm	
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 501.96 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 2 bolts = 406.66 kN	
Cl.6.2.6.5 (1)	Bolt row 3 - Other end bolt row	
T 6.6	Effective length for an end plate,	
	for circular patterns, $\ell_{eff,cp} = 2\pi m$	
	m = 38.55 mm	
	$\ell_{eff,cp}$ = 242.09 mm	
	for non circular patterns, $\ell_{eff,nc} = 4m + 1.25e$	
	= 247.95 mm	
	$\ell_{eff,nc}$ = 247.95 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 242.09 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	n = 48.19 mm	
	$M_{p,1,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_p^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 242.09 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 10024.2 kNmm	
	$F_{T,1,Rd}$ = 1282.91 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 247.95 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	

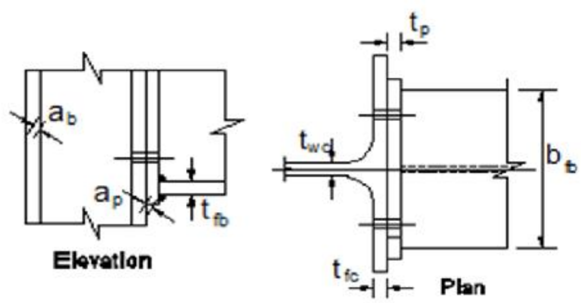
Reference	Calculation	Output
	$M_{p,2,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_p^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 247.95 * 25^2 * 265}{1.0}$	
	$M_{p\,2,Rd}$ = 10266.7 kNmm	
	n = e_{min} but n ≤ 1.25m (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 462.65 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 3 bolts = 406.66 kN	
Cl.6.2.6.5 (1)	Bolt row 2 & 3 - combined	
	resistance of row 3 may be limited by the resistance of rows 2 & 3 as a group	
	row 2 first bolt row below the tension flange of the beam	
	row 3 other end bolt row	
T 6.6	Effective length for an end plate,	
	row 2	
	for circular patterns, $\ell_{eff,cp} = \pi m + p$	
	m = 38.55 mm p = 90 mm	
	$\ell_{eff,cp}$ = 211.05 mm	
	for non circular patterns, $\ell_{eff,nc}$ = 0.5p + αm-(2m + .625e)	
Figure 6.11	$\lambda_1 = \dfrac{m}{m+e} = 0.34$, $\lambda_2 = \dfrac{m_2}{m+e} = 0.31$	
	α = 7.5	
	$\ell_{eff,nc}$ = 210.15 mm	
	row 3	
	for circular patterns, $\ell_{eff,cp} = \pi m + p$	
	m = 38.55 mm p = 90 mm	
	$\ell_{eff,cp}$ = 211.05 mm	
	for non circular patterns, $\ell_{eff,nc}$ = 2m + .625e+0.5*p	
	= 169 mm	
	total effective length for this group of rows	
	Σℓeff,cp = 211 + 211 = 422 mm	
	Σℓeff,nc = 210 + 169 = 379 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \le \ell_{eff,cp}$	
	$\Sigma\ell_{eff,1}$ = 379.13 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	n = e_{min} but n ≤ 1.25m (48.2)	
	n = 48.19 mm	

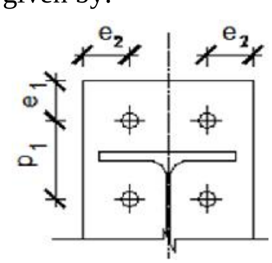
Reference	Calculation	Output
	$M_{p,1,Rd} = \frac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 379.13 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 15698.1 kNmm	
	$F_{T,1,Rd}$ = 2009.07 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 379.13 mm	
T 6.2	$F_{T,2,Rd} = \frac{2 M_{p,2,Rd} + n \sum F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \frac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \frac{0.25 * 379.13 * 25^2 * 265}{1.0}$	
	$M_{p\ 2,Rd}$ = 15698.1 kNmm	
	$n = e_{min}$ but $n \le 1.25m$ (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 813.81 kN	
	Mode 3	
	$F_{T,3,Rd} = \sum F_{t,Rd}$ = 4 * 203.33 = 813.312 kN	
	Resistance only from 2 & 3 bolts = 813.31 kN	F_2 =
	Resistance for End plate in bending = 377.26 kN	377.26 kN
Cl.6.2.6.4	**3.Column flange in transverse bending**	
Cl.6.2.6.4 (1)	- each individual bolt-row required to resist tension	
Cl.6.2.6.4 (1)	Bolt row 1 - end bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\ell_{eff,cp}$ = smaller of $2\pi m$ and $\pi m + 2 e_1$	
	for welded end plate narrower than column flange	
	r_c = 12.7 mm	
	m = 33.44 mm	
	e = 79.4 mm	
	e_1 is large so it will not be critical	
	e_{min} = 75 mm	
	$2\pi m$ = 210.003 mm	
	$\ell_{eff,cp}$ = 210.00 mm	
	for non circular patterns,	
	$\ell_{eff,nc}$ = smaller of $4m + 1.25\ e$ and $2m + 0.625\ e + e_1$	
	$4m + 1.25\ e$ = 233.01 mm	
	$\ell_{eff,nc}$ = 233.01 mm	
	Mode 1	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \le \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 210.00 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	$n = e_{min}$ but $n \le 1.25m$ (41.8)	
	n = 41.80 mm	
	$M_{p,1,Rd} = \frac{0.25 \sum \ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \frac{0.25 * 210.00 * 20.5^2 * 265}{1.0}$	

Reference	Calculation	Output
	$M_{p,1,Rd}$ = 5846.8 kNmm $F_{T,1,Rd}$ = 896.62 kN	
	Mode 2	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 233.01 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	
	$M_{p,2,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 233.01 * 20.5^2 * 265}{1.0}$ $M_{p\,2,Rd}$ = 6487.4 kNmm	
	n = e_{min} but n ≤ 1.25m (41.8) n = 41.8 mm $F_{t,Rd}$ = 203.33 $F_{T,2,Rd}$ = 398.36 kN	
	Mode 3 $F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 1 bolts = 398.36 kN	
Cl.6.2.6.4 (1)	Bolt row 1 and 2 combined Bolt row 1 - end bolt row Bolt row 2 - end bolt row	
T 6.4	Effective length of an unstiffened column flange for circular patterns, $\ell_{eff,cp} = (\pi m + p)$ for welded end plate narrower than column flange r_c = 12.7 mm m = 33.44 mm e = 79.4 mm p = 100 mm	
	$2(\pi m + p)$ = 410.003 mm $\Sigma\ell_{eff,cp}$ = 410.00 mm	
	for non circular patterns, $\ell_{eff,nc}$ = 2*(2m + 0.625e + 0.5p) 2(2m + 0.625e + 0.5p) = 333.01 mm $\Sigma\ell_{eff,nc}$ = 333.01 mm	
	Mode 1 $\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$ $\Sigma\ell_{eff,1}$ = 333.01 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$ e_w = 9.8875 mm	
	$M_{p,1,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 333.01 * 20.5^2 * 265}{1.0}$ $M_{p,1,Rd}$ = 9271.52 kNmm $F_{T,1,Rd}$ = 1421.81 kN	

Reference	Calculation	Output
	Mode 2	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc} = 333.01$ mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	
	$M_{p,2,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 333.01 * 20.5^2 * 265}{1.0}$	
	$M_{p\,2,Rd} = 9271.52$ kNmm	
	$n = e_{min}$ but $n \leq 1.25m$	
	$n = 41.8$ mm	
	$F_{t,Rd} = 203.33$	
	$F_{T,2,Rd} = 698.29$ kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd} = 4 * 203.33 = 813.312$ kN	
	Resistance from Bolt Row 1 & Row 2 combination = 698.29 kN	
Cl.6.2.6.4 (1)	Bolt row 1,2 and 3 combined	
	Bolt row 1 ,3 - end bolt row	
	Bolt row 2 - inner bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\Sigma\ell_{eff,cp} = 2p + (\pi m + p1)+(\pi m + p2)$	
	for welded end plate narrower than column flange	
	$r_c = 12.7$ mm	
	$m = 33.44$ mm	
	$e = 79.4$ mm	
	$p1 = 100$ mm $p = 95$ mm	
	$p2 = 90$ mm	
	$\Sigma\ell_{eff,cp} = 2p + (\pi m + p1)+(\pi m + p2)$	
	$= 590.003$ mm	
	for non circular patterns,	
	$\Sigma\ell_{eff,nc} = p + (2m + 0.625e + 0.5p1) + (2m + 0.625e + 0.5p2)$	
	$\Sigma\ell_{eff,nc} = 423.01$ mm	
	Mode 1	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\Sigma\ell_{eff,1} = 423.01$ mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	$e_w = 9.8875$ mm	
	$M_{p,1,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 423.01 * 20.5^2 * 265}{1.0}$	
	$M_{p,1,Rd} = 11777.3$ kNmm	
	$F_{T,1,Rd} = 1806.07$ kN	
	Mode 2	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc} = 423.01$ mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$	

Reference	Calculation	Output
	$M_{p,2,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25*\ 423.01\ *\ 20.5^2 *\ 265}{1.0}$	
	$M_{p\,2,Rd}$ = 11777.3 kNmm	
	$n = e_{min}$ but $n \leq 1.25m$	
	n = 41.8 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 990.82 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 6 * 203.33 = 1219.97 kN	
	Resistance from Bolt Row 1,2 & 3 combination = 990.82 kN	
Cl.6.2.6.4 (1)	Bolt row 2 and 3 combined	
	Bolt row 2,3 - end bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\Sigma\ell_{eff,cp} = 2*(\pi m + p)$	
	for welded end plate narrower than column flange	
	r_c = 12.8 mm	
	m = 33.44 mm	
	e = 79.4 mm	
	p = 90 mm	
	$\Sigma\ell_{eff,cp}$ = 390.003 mm	
	for non circular patterns,	
	$\ell_{eff,nc} = 2*(2m + 0.625e + 0.5p)$	
	$\Sigma\ell_{eff,nc}$ = 323.01 mm	
	Mode 1	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\Sigma\ell_{eff,1}$ = 323.01 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 9.8875 mm	
	$M_{p,1,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25*\ 323.01\ *\ 20.5^2 *\ 265}{1.0}$	
	$M_{p,1,Rd}$ = 8993.10 kNmm	
	$F_{T,1,Rd}$ = 1379.11 kN	
	Mode 2	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 323.01 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2\,M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_f^2\,f_y}{\gamma_{MO}} = \dfrac{0.25*\ 323.01\ *\ 20.5^2 *\ 265}{1.0}$	
	$M_{p\,2,Rd}$ = 8993.10 kNmm	
	$n = e_{min}$ but $n \leq 1.25m$	
	n = 41.8 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 690.89 kN	

Reference	Calculation	Output
	Mode 3 $F_{T,3,Rd} = \Sigma F_{t,Rd} = 4 * 203.33 = 813.312$ kN	
	Resistance from Bolt Row 2 & 3 combination = 690.89 kN	
	Resistance for Column flange in transverse bending = 398.36 kN	F_3 = 398.36 kN
Cl.6.2.6.8 (1) Eq" (6.22)	**4. Beam web in tension** $F_{t,wb,Rd} = \frac{b_{eff,t,wb}\ t_{wb}\ f_{y,wb}}{\gamma_{Mo}}$ $b_{eff,t,wb}$ = 242 mm t_{wb} = 10.1 mm $F_{t,wb,Rd}$ = 672.42 kN	F_4 = 672.42 kN
EN 1993-1-8 :2005 Cl.6.2.6.3 (1) Cl.6.2.6.3 (3)	**5. Column web in tension** $F_{t,wc,Rd} = \frac{\omega\ b_{eff,t,wc}\ t_{wc}\ f_{y,wc}}{\gamma_{Mo}}$ For Bolted connection $b_{eff,t,wc}$ = 233.00 mm t_{wc} = 12.8 mm ω = 1.00 $F_{t,wc,Rd}$ = 790.34 kN	F_5 = 790.34 kN
Cl.6.2.6.1	**6. Column web panel in shear** $V_{wp,Rd} = \frac{0.9\ f_{y,wc}\ A_{vc}}{\sqrt{3}\ \gamma_{Mo}}$ A_{vc} = 267 * 12.8 = 3413.76 mm^2 $V_{wp,Rd}$ = 470.068 kN	F_6 = 470.068 kN
EN 1993-1-8 :2005 Cl.6.2.6.2 T 5.4 T 6.3	**7,8. Column web in compression** $F_{c,wc,Rd} = \frac{\omega\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{y,wc}}{\gamma_{Mo}} \leq \frac{\omega\ k_{wc}\ \rho\ b_{eff,c,wc}\ t_{wc}\ f_{y,wc}}{\gamma_{M1}}$ Transformation factor, $\beta \approx 1$ $\omega = \omega_1 = \frac{1}{[1 + 1.3\ (b_{eff,c,wc}\ t_{wc} / A_{vc})^2]^{1/2}}$ For bolted end plate connection $b_{eff,c,wc} = t_{fb} + 2\sqrt{2}\ a_p + 5(t_{fc} + s) + s_p$	
Figure 6.6	 t_{fb} = 15.6 mm a_p = 8.4 mm t_{fc} = 20.5 mm t_p = 25.0 mm t_{wc} = 12.8 mm	
	s_p = 2 * t_p = 50.0 mm For a rolled I or H section column, s = r_c = 12.7 mm $b_{eff,c,wc}$ = 248.4 mm	

Reference	Calculation	Output
	A_{vc} = 3414 mm^2	
	$\omega = \omega_1 = 0.69$	
Cl.6.2.6.2 (2)	Assume longitudinal compressive stress, $\sigma_{com,Ed} < 0.7 f_{y,wc}$	
	k_{wc} = 1.0	
	t_{wc} = 12.8 mm	
	$\dfrac{\omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{Mo}}$ = 842573 N	
	Column web Bearing resistance = 842.573 kN	F_7 = 842.57 kN
Eq^n (6.13c)	$\bar{\lambda}p = 0.932 \left(\dfrac{b_{eff,c,wc}\, d_{wc}\, f_{y,wc}}{E\, t_{wc}^2}\right)^{1/2}$	
	$b_{eff,c,wc}$ = 248.4 mm	
	for rolled I or H section column: $d_{wc} = h_c - 2\,(t_{fc} + r_c)$	
	h_c = 267 mm	
	t_{fc} = 20.5 mm	
	r_c = 12.7 mm	
	d_{wc} = 200.30 mm	
	E = 210.0 kN/mm^2	
	$\bar{\lambda}p$ = 0.58	
	$\bar{\lambda}p$ < 0.72	
	ρ = Buckling reduction factor = 1	
	$\dfrac{\omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M1}}$ = 842573 N	
	Column web Buckling resistance = 842.573 kN	F8 = 842.57 kN
EN 1993-1-8 :2005 T 3.4	**9. Bolt Shear**	
	Resistance of a single bolt in shear ($F_{v,Rd}$) is given by:	
	$F_{v,Rd} = \dfrac{\alpha_v f_{ub} A}{\gamma_{M2}}$	
	Where; α_v = 0.6 for class 8.8 bolts	
	A = A_s = 353 mm^2	
	$F_{v,Rd} = \dfrac{0.6 \times 800 \times 353}{1.25} \times 10^{-3}$ = 135.552 kN	
	$V_{Rd} = n\, F_{v,Rd}$	
	No: of Bolts in Shear = 8	
	Shear Resistance of the connection = 1084.42 kN	F_9 = 1084.42 kN
EN 1993-1-8 :2005 T 3.4	**10. Bolt Bearing**	
	The bearing Resistance of a single bolt ($F_{b,Rd}$) is given by:	
	$F_{b,Rd} = \dfrac{k_1\, \alpha_b\, f_{ub}\, d\, t_p}{\gamma_{M2}}$	
	Where α_b is the least value of α_d, $\dfrac{f_{ub}}{f_{y,p}}$ and 1 	
	For the Direction of load transfer	
	For end Bolts $\alpha_d = e_1$ = 50 = 0.64	

Reference	Calculation	Output

For inner Bolts $\alpha_d = \dfrac{p_1}{3d_0} - \dfrac{1}{4} = \dfrac{100}{3x\ \ 26} - \dfrac{1}{4} = 1.03$

$\dfrac{f_{ub}}{f_{u,p}} = \dfrac{800}{410} = 1.95$

$\alpha_b = 0.64$

For the perpendicular to the Direction of load transfer

For edge bolts k1, is the smaller of $2.8\dfrac{e_2}{d_0} - 1.7$ or 2.5

$2.8\dfrac{e_2}{d_0} - 1.7 = 2.8 \times \dfrac{75}{26} - 1.7 = 7.85$

Therefore for edge bolts, $k_1 = 2.50$

For inner bolts k1, is the smaller of $1.4\dfrac{p_2}{d_0} - 1.7$ or 2.5

$1.4\dfrac{p_2}{d_0} - 1.7 = 1.4 \times \dfrac{100}{26} - 1.7 = 3.68$

Therefore for inner bolts, $k_1 = 2.50$

Therefore the minimum bearing resistance for a bolt is:

$F_{b,Rd} = \dfrac{2.50 \times 0.64 \times 410 \times 24 \times 25}{1.10}$

$= 358.392$ kN

bearing resistance of the connection: $= 8 * 358.392$

$= 2867.13$ kN

Output: $F_{10} = 2867.13$ kN

Cl.6.2.6.7

11. Beam flange and web in compression

Eq^n (6.21)

$F_{c,fb,Rd} = M_{c,Rd} / (h - t_{fb})$

$M_{c,Rd}$ = Design resistance of the beam

assume that the design shear force in the beam doesn't reduce $M_{c,Rd}$

therefore, from P363

$M_{c,Rd} = 649$ kNm

$F_{c,fb,Rd} = \dfrac{649}{533 - 15.6} = 1254.11$ kN

Output: $F_{11} = 1254.11$ kN

Reference	Calculation	Output

Summary of tension resistance

	Column flange bending	Column web in tension	End plate in bending	beam web in tension	minimum	effective resistance
Row 1, alone	398.36	790	377.26	N/A	377.26	377.26
Row 2,alone	398.36	790	406.66	672.42	398.36	
Row 2, with row 1	698.29		N/A	N/A	698.29	
Row 2					321.03	321.03
Row 3, alone	398.36	790	406.66	672.42	398.36	
Row 3, with row 1 &2	990.82		N/A	N/A	990.82	
Row 3					292.53	
Row 3, with row 2	690.89		813.31		690.89	
Row 3					369.86	292.53

Column web in Transverse compression = 842.57 kN

Beam flange and web in compression is not critical

Moment resistance

Effective resistance of bolt rows

The effective resistance of each of the three bolt rows in tension zone

$F_{t1,Rd}$ = 377.26 kN

$F_{t2,Rd}$ = 321.03 kN

$F_{t3,Rd}$ = 292.53 kN

Effective resistance should be reduced if the resistance of one of the higher rows exceeds

1.9 x F t,Rd = 386.323 kN

Resistance of bolt row 1 & 2 are less than this value.
Hence no reduction is required

Reference	Calculation	Output
	Total effective tension resistance	
	$\sum F_{t,Rd}$ = 377.26 + 321.03 + 292.53	
	= 990.82 kN	
	Compression resistance = 842.57 kN	
	Here, total tension resistance exceeds the compression resistance	
	reduction required = 148.25 kN	
	$F_{t3,Rd}$ = 144.28 kN	
	Moment resistance of the beam to column joint	
	= 565.3 x 377.26 + 465.3 x 321.03 + 375.3 x 144.28	
	= 417 kNm	
	M_p = Design plastic moment resistance of beam	
	$M_p = (p_y . Z)/\gamma_{m0}$	
	$= \dfrac{275 \times 2072 \times 1000}{1.0}$	
EN1993-1-8 :2005 CL5.2.3	= 569.8 kNm	
	$M_{con}/M_{p,\ beam}$ = 0.73147	
	Hence semi rigid connection	

Reference	Calculation	Output
Table 6.10 Table 6.11 BS EN1993-1-8 :2005	**Determination of rotational stiffness for EEP 3d** **Stiffness coefficient** **1.Column web panel in shear** $k1 = (0.38\, A_{vc})/\beta Z$ Z = Lever arm β = Transformation parameter A_{vc} = Shear area of the column $k1 = (0.38\, X\, (266.7x12.8))/(1X\, (670 - 50 - (40+60)/2 - 25 - 15.6/2))$ = 2.41 **2.Column web in compression** $k2 = (0.7\, b_{eff,c,wc}\, X\, t_{wc})/d_c$ $b_{eff,c,wc}$ = effective width t_{wc} = column web thickness d_c = clear depth of column $k2 = (0.7\, X 248.4\, X 12.8)/200.3$ k2 = 11.11 **3. Column web in tension** $k_{3,1} = (0.7\, X b_{eff,t1wc}\, X\, t_{wc})/d_c$ $k_{3,1} = (0.7\, X\, 166.5\, X\, 12.8)/200.3$ $k_{3,1}$ = 7.45 $k_{3,2} = (0.7\, X\, 95\, X\, 12.8)/200.3$ $k_{3,2}$ = 4.25 $k_{3,3} = (0.7\, X\, 161.5\, X\, 12.8)/200.3$ $k_{3,3}$ = 7.22 Contribution from bolt row 4 is neglected.	

Reference	Calculation	Output
	4. Column flange in bending	
	$k_{4,1}=(0.9\ X\ l_{eff}\ X\ t_{fc}{}^{3})/m^{3}$	
	$k_{4,1}=(0.9\ X\ 166.5\ X20.5^{3})/33.44^{3}$	
	$k_{4,1} = 34.52$	
	$k_{4,2}=(0.9\ X\ 95\ X20.5^{3})/33.44^{3}$	
	$k_{4,2} = 19.70$	
	$k_{4,3}=(0.9\ X\ 161.5\ X20.5^{3})/33.44^{3}$	
	$k_{4,3} = 33.49$	
	5. End plate bending	
	$k_{5,1}=(0.9\ X\ l_{eff}\ X\ t_{p}{}^{3})/m^{3}$	
	$k_{5,1}=(0.9\ X\ 125\ \ X\ 25^{3})/30.4^{3}$	
	$k_{5,1} = 62.57$	
	$k_{5,2}=(0.9\ X\ 210.15\ \ X\ 25^{3})/38.55^{3}$	
	$k_{5,2} = 51.58$	
	$k_{5,3}=(0.9\ X\ 169\ \ X\ 25^{3})/38.55^{3}$	
	$k_{5,3} = 41.48$	
	6. Bolts in tension	
	$k_{10}=(1.6\ A_{S})/L_{b}$	
	L_b - Bolt elongation length	
	L_b = 48.5	
	k_{10} = 11.65	

Reference	Calculation	Output
BS EN1993-1-8 :2005 CL 6.3.3.1	The effective stiffness coefficient of each bolt row is obtained as follows. $k_{eff,1}=(1/((1/k_{3,1})+(1/k_{4,1})+(1/k_{5,1})+(1/k_{10}))$ $k_{eff,1}=(1)/((1/7.45)+(1/34.52)+(1/62.57)+(1/11.65))$ = 3.772 $k_{eff,2}=(1)/((1/4.25)+(1/19.7)+(1/51.58)+(1/11.65))$ = 2.555 $k_{eff,2}=(1)/((1/7.22)+(1/33.49)+(1/41.48)+(1/11.65))$ = 3.594 $h_1=670-25-15.6/2-C$ h_1 = 587.2 mm $h_2=670-25-15.6/2-c-(d+e)$ h_2 = 487.2 mm $h_3=670-25-15.6/2-c-(d+e)-90$ h_3 = 397.2 mm	
BS EN1993-1-8 :2005 CL 6.3.3.1	Equivalent lever arm Zeq is Z_{eq} = 506.24 mm k_{eq} = 9.65	
BS EN1993-1-8 :2005 CL 6.3.1	**The initial joint stiffness** $S_{j,ini}=(E\,Z^2)/(\mu\sum(1/ki))$ μ = 1 $S_{j,ini}=(210x10^3x506.24^2)/((1/9.65)+(1/2.41)+(1/11.11))$ = 8.86E+07 kNmm / rad	

Reference	Calculation	Output
BS EN1993-1-8 :2005 CL5.2.2.5 figure 5.4	$EI_b/L_b = 210x10^3 x55227x10^4/6000x10^3$ = 1.93E+07 kNmm / rad I_b - second moment of area L_b - span of beam $0.5E * I_b/L_b < S_{j,ini} < 8*E*I_b/L_b$ $S_{j,ini}/EI_b/L_b =$ 4.58 Hence semi rigid connection	

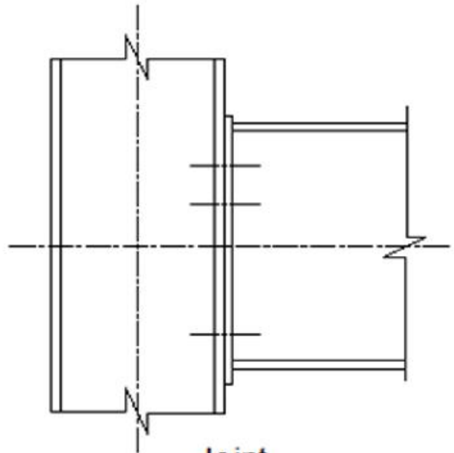
L	EIb/Lb	$S_{j,ini}/EI_b/L_b$	Classification
4000	28994175	3.05	semi rigid
6000	19329450	4.58	semi rigid
8000	14497087.5	6.11	semi rigid
10000	11597670	7.64	semi rigid
12000	9664725	9.16	rigid

Appendix B:

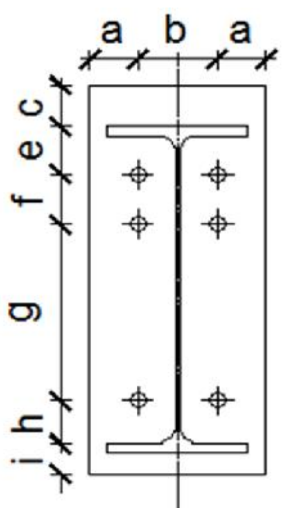
Design Calculations for Flush End Plate Connection- FEP 3d

Reference	Calculation	Output

Connection Detail - FEP 3d



Joint



Row 1

Row 2

a	=	75	mm
b	=	100	mm
c	=	25	mm
e	=	60	mm
f	=	90	mm
g	=	345	mm
h	=	60	mm
i	=	25	mm

Beam 533x210x92

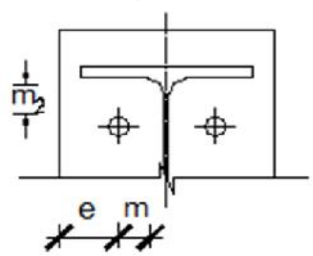
Beam height	=	533.1	mm
Flange width	=	209.3	mm
Mass per 1m length	=	92	kg/m
Flange thickness	=	15.6	mm
Web thickness	=	10.1	mm
Radius of gyration	=	12.7	mm

Column 254x254x107

column height	=	266.7	mm
Flange width	=	258.8	mm
Mass per 1m length	=	107	kg/m
Flange thickness	=	20.5	mm
Web thickness	=	12.8	mm
Radius of gyration	=	12.7	mm

Yield strength	$f_{y,p}$	=	265	N/mm^2
Ultimate tensile strength	$f_{u,p}$	=	410	N/mm^2

Reference	Calculation	Output
	End plate	
	End plate thickness t_p = 25 mm	
	End plate height h_p = 605 mm	
	End plate width w_p = 250 mm	
	Yield strength $f_{y,p}$ = 265 N/mm^2	
	Ultimate tensile strength $f_{u,p}$ = 410 N/mm^2	
	Bolts (class 8.8)	
	Bolt Diameter = 24 mm d_w = 40 mm	
	Tensile stress area = 353 mm^2	
	Total no of bolts = 6	
	no of bolts in tension = 4	
	no of bolts in shear = 6	
T 3.1	Yield strength f_{yb} = 640 N/mm^2	
	Ultimate tensile strength f_{ub} = 800 N/mm^2	
	fillet weld thickness	
	Beam flange to end plate weld thickness = 12 mm	
	Beam web to end plate weld thickness = 8 mm	

Reference	Calculation	Output
	Design Calculation according to EC3 for FEP	
	Partial factors for Resistance	
	Structural Steel	
EN 1993-1-8	γ_{MO} = 1.0	
N.A.2.15		
T NA.1	γ_{M1} = 1.00 (Resistance of a member to buckling)	
T NA.1	γ_{M2} = 1.10 (plates in bearing in bolted connections)	
	For tring resistance verification $\gamma_{M,u}$ = 1.10	
T NA.1	**Bolts** γ_{M2} = 1.25	
T NA.1	**Welds** γ_{M2} = 1.25	
EN 1993-1-8 :2005	**1. Bolts Tension**	
Cl.3.6.1 (1) T 3.4	$F_{t,Rd} = \dfrac{k_2 f_{ub} A_s}{\gamma_{M2}}$	
	For non countersunk Bolts , k_2 = 0.9	F_1 =
	$F_{t,Rd} = \dfrac{k_2 f_{ub} A_s}{\gamma_{M2}} = \dfrac{0.9 \times 800 \times 353}{1.25}$ = 203.328 kN	203.33 kN
Cl.6.2.6.5	**2. End plate in bending**	
	for flush end plate	
	w = 100 mm	
	e = 75 mm	
	e_{min} = 75 mm	
		
Cl.6.2.6.5 (1)	Bolt row 1 - First Bolt row below tension flange of beam	
T 6.6	Effective length for an end plate,	
	for circular patterns, $\ell_{eff,cp}$ = $2\pi m$	
	m = 38.55 mm	
	$\ell_{eff,cp}$ = 242.09 mm	
	for non circular patterns, $\ell_{eff,nc}$ = αm	
	m_2 = 34.8 mm	
Figure 6.11	$\lambda_1 = \dfrac{m}{m+e}$ = 0.34 , $\lambda_2 = \dfrac{m_2}{m+e}$ = 0.31	
	α = 7.5	
	$\ell_{eff,nc}$ = 289.13 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1}$ = $\ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 242.09 mm	
T 6.2	$F_{T,1,Rd}$ = $(8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	n = e_{min} but n $\leq$ 1.25m (48.2)	
	n = 48.19 mm	
	$M_{p,1,Rd} = \dfrac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 242.09 * 25^2 * 265}{1.0}$	

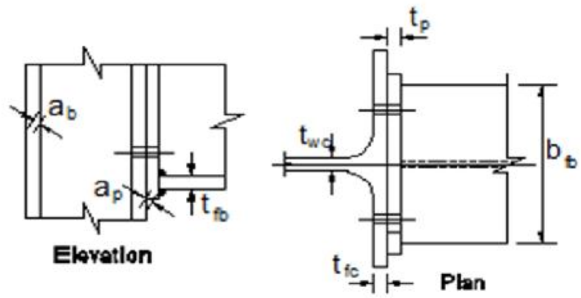
Reference	Calculation	Output
	$M_{p,1,Rd}$ = 10024.2 kNmm	
	$F_{T,1,Rd}$ = 1287 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc} = 289.13$ mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2 M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \dfrac{0.25 \Sigma\ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 289.13 * 25^2 * 265}{1.0}$	
	$M_{p\,2,Rd}$ = 11971.6 kNmm	
	n = e_{min} but n ≤ 1.25m (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 501.96 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 1 bolts = 406.66 kN	
Cl.6.2.6.5 (1)	Bolt row 2 - Other end bolt row	
T 6.6	Effective length for an end plate,	
	for circular patterns, $\ell_{eff,cp} = 2\pi m$	
	m = 38.55 mm	
	$\ell_{eff,cp}$ = 242.09 mm	
	for non circular patterns, $\ell_{eff,nc}$ = 4m + 1.25e	
	= 247.95 mm	
	$\ell_{eff,nc}$ = 247.95 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \le \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 242.09 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	n = e_{min} but n ≤ 1.25m (48.2)	
	n = 48.19 mm	
	$M_{p,1,Rd} = \dfrac{0.25 \Sigma\ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 242.09 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 10024.2 kNmm	
	$F_{T,1,Rd}$ = 1286.52 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc} = 247.95$ mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2 M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \dfrac{0.25 \Sigma\ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 247.95 * 25^2 * 265}{1.0}$	
	$M_{p\,2,Rd}$ = 10266.7 kNmm	

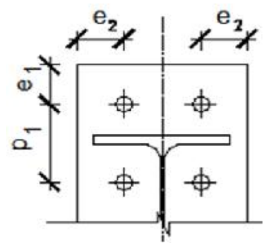
Reference	Calculation	Output
	$n = e_{min}$ but $n \le 1.25m$ (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 462.65 kN	
	Mode 3	
	$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 2 bolts = 406.66 kN	
Cl.6.2.6.5 (1)	Bolt row 1 & 2 - combined	
	row 1 first bolt row below the tension flange of the beam	
	row 2 other end bolt row	
T 6.6	Effective length for an end plate,	
	row 1	
	for circular patterns, $\ell_{eff,cp} = \pi m + p$	
	m = 38.55 mm p = 90 mm	
	$\ell_{eff,cp}$ = 211.05 mm	
	for non circular patterns, $\ell_{eff,nc} = 0.5p + \alpha m - (2m + .625e)$	
Figure 6.11	$\lambda_1 = \dfrac{m}{m + e}$ = 0.34 , $\lambda_2 = \dfrac{m_2}{m + e}$ = 0.31	
	α = 7.5	
	$\ell_{eff,nc}$ = 210.15 mm	
	row 2	
	for circular patterns, $\ell_{eff,cp} = \pi m + p$	
	m = 38.55 mm p = 90 mm	
	$\ell_{eff,cp}$ = 211.05 mm	
	for non circular patterns, $\ell_{eff,nc} = 2m + .625e + 0.5*p$	
	= 169 mm	
	total effective length for this group of rows	
	$\Sigma\ell eff,cp$ = 211 + 211 = 422 mm	
	$\Sigma\ell eff,nc$ = 210 + 169 = 379 mm	
T 6.2	**Mode 1** - Complete failure of the T-stub flange	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \le \ell_{eff,cp}$	
	$\Sigma\ell_{eff,1}$ = 379.13 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	$n = e_{min}$ but $n \le 1.25m$ (48.2)	
	n = 48.19 mm	
	$M_{p,1,Rd} = \dfrac{0.25\,\Sigma\ell_{eff}\,t_p^2\,f_y}{\gamma_{MO}} = \dfrac{0.25 * 379.13 * 25^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 15698.1 kNmm	

Reference	Calculation	Output
	$F_{T,1,Rd}$ = 2014.72 kN	
	Mode 2 - Bolt failure with yielding of the T-stub flange	
T 6.4	$\ell_{eff,2} = \ell_{eff,nc}$ = 379.13 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2 M_{p,2,Rd} + n\sum F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \dfrac{0.25 \sum \ell_{eff} t_p^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 379.13 * 25^2 * 265}{1.0}$	
	$M_{p\ 2,Rd}$ = 15698.1 kNmm	
	$n = e_{min}$ but $n \leq 1.25m$ (48.2)	
	n = 48.1875 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 813.81 kN	
	Mode 3	
	$F_{T,3,Rd} = \sum F_{t,Rd}$ = 4 * 203.33 = 813.312 kN	
	Resistance only from 1 & 2 bolts = 813.31 kN	
	Resistance for End plate in bending = 406.66 kN	F_2 = 406.66 kN
Cl.6.2.6.4	**3.Column flange in transverse bending**	
Cl.6.2.6.4 (1)	- each individual bolt-row required to resist tension	
Cl.6.2.6.4 (1)	Bolt row 1 - end bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\ell_{eff,cp}$ = smaller of $2\pi m$ and $\pi m + 2 e_1$	
	for welded end plate narrower than column flange	
	r_c = 12.7 mm	
	m = 33.44 mm	
	e = 79.4 mm	
	e_1 is large so it will not be critical	
	e_{min} = 75 mm	
	$2\pi m$ = 210.003 mm	
	$\ell_{eff,cp}$ = 210.00 mm	
	for non circular patterns,	
	$\ell_{eff,nc}$ = smaller of 4m + 1.25 e and 2m + 0.625 e + e_1	
	4m + 1.25 e = 233.01 mm	
	$\ell_{eff,nc}$ = 233.01 mm	
	Mode 1	
	$\ell_{eff,1} = \ell_{eff,nc}$ but $\ell_{eff,1} \leq \ell_{eff,cp}$	
	$\ell_{eff,1}$ = 210.00 mm	
T 6.2	$F_{T,1,Rd} = (8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	$n = e_{min}$ but $n \leq 1.25m$ (41.8)	
	n = 41.80 mm	
	$M_{p,1,Rd} = \dfrac{0.25 \sum \ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \dfrac{0.25* 210.00 * 20.5^2 * 265}{1.0}$	
	$M_{p,1,Rd}$ = 5846.8 kNmm	
	$F_{T,1,Rd}$ = 899.69 kN	

Reference	Calculation	Output
	Mode 2	
T 6.4	$\ell_{eff,2}$ = $\ell_{eff,nc}$ = 233.01 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2 M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m + n}$	
	$M_{p,2,Rd} = \dfrac{0.25 \Sigma\ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \dfrac{0.25* \quad 233.01 \quad * \quad 20.5^2 * \quad 265}{1.0}$	
	$M_{p\,2,Rd}$ = 6487.4 kNmm	
	n = e_{min} but n ≤ 1.25m (41.8)	
	n = 41.8 mm	
	$F_{t,Rd}$ = 203.33	
	$F_{T,2,Rd}$ = 398.36 kN	
	Mode 3	
	$F_{T,3,Rd}$ = $\Sigma F_{t,Rd}$ = 2 * 203.33 = 406.656 kN	
	Resistance only from Row 1 bolts = 398.36 kN	
Cl.6.2.6.4 (1)	Bolt row 1 and 2 combined	
	Bolt row 1,2 - end bolt row	
T 6.4	Effective length of an unstiffened column flange	
	for circular patterns, $\Sigma\ell_{eff,cp}$ = 2*(πm + p)	
	for welded end plate narrower than column flange	
	r_c = 12.8 mm	
	m = 33.44 mm	
	e = 79.4 mm	
	p = 90 mm	
	$\Sigma\ell_{eff,cp}$ = 390.003 mm	
	for non circular patterns,	
	$\ell_{eff,nc}$ = 2*(2m + 0.625e + 0.5p)	
	$\Sigma\ell_{eff,nc}$ = 323.01 mm	
	Mode 1	
	$\ell_{eff,1}$ = $\ell_{eff,nc}$ but $\ell_{eff,1}$ ≤ $\ell_{eff,cp}$	
	$\Sigma\ell_{eff,1}$ = 323.01 mm	
T 6.2	$F_{T,1,Rd}$ = $(8n-2e_w)M_{p,1,Rd}/[2mn-e_w(m+n)]$	
	e_w = 10 mm	
	$M_{p,1,Rd} = \dfrac{0.25 \Sigma\ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \dfrac{0.25* \quad 323.01 \quad * \quad 20.5^2 * \quad 265}{1.0}$	
	$M_{p,1,Rd}$ = 8993.10 kNmm	
	$F_{T,1,Rd}$ = 1383.84 kN	
	Mode 2	
T 6.4	$\ell_{eff,2}$ = $\ell_{eff,nc}$ = 323.01 mm	
T 6.2	$F_{T,2,Rd} = \dfrac{2 M_{p,2,Rd} + n\Sigma F_{t,Rd}}{m + n}$	

Reference	Calculation	Output
	$M_{p,2,Rd} = \dfrac{0.25 \sum \ell_{eff} t_f^2 f_y}{\gamma_{MO}} = \dfrac{0.25 * 323.01 * 20.5^2 * 265}{1.0}$	
	$M_{p2,Rd} = 8993.10$ kNmm	
	$n = e_{min}$ but $n \leq 1.25m$	
	$n = 41.8$ mm	
	$F_{t,Rd} = 203.33$	
	$F_{T,2,Rd} = 690.89$ kN	
	Mode 3	
	$F_{T,3,Rd} = \sum F_{t,Rd} = 4 * 203.33 = 813.312$ kN	
	Resistance from Bolt Row 1 & 2 combination = 690.89 kN	
	Resistance for Column flange in transverse bending = 398.36 kN	F_3 = 398.36 kN
Cl.6.2.6.8 (1) Eq" (6.22)	**4. Beam web in tension**	
	$F_{t,wb,Rd} = \dfrac{b_{eff,t,wb} t_{wb} f_{y,wb}}{\gamma_{Mo}}$	
	$b_{eff,t,wb} = 242$ mm	
	$t_{wb} = 10.1$ mm	
	$F_{t,wb,Rd} = 672.42$ kN	F_4 = 672.42 kN
EN 1993-1-8 :2005	**5. Column web in tension**	
Cl.6.2.6.3 (1)	$F_{t,wc,Rd} = \dfrac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{Mo}}$	
Cl.6.2.6.3 (3)	For Bolted connection	
	$b_{eff,t,wc} = 210.00$ mm	
	$t_{wc} = 12.8$ mm	
	$\omega = 1.00$	F_5 =
	$F_{t,wc,Rd} = 712.33$ kN	712.33 kN
Cl.6.2.6.1	**6. Column web panel in shear**	
	$v_{wp,Rd} = \dfrac{0.9 f_{y,wc} A_{vc}}{\sqrt{3} \gamma_{Mo}}$	
	$A_{vc} = 267 * 12.8 = 3413.76$ mm^2	F_6 =
	$v_{wp,Rd} = 470.068$ kN	470.068 kN
EN 1993-1-8 :2005	**7,8. Column web in compression**	
Cl.6.2.6.2	$F_{c,wc,Rd} = \dfrac{\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{Mo}} \leq \dfrac{\omega k_{wc} \rho b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M1}}$	
T 5.4	Transformation factor, $\beta \approx 1$	
T 6.3	$\omega = \omega_1 = \dfrac{1}{[1 + 1.3 (b_{eff,c,wc} t_{wc} / A_{vc})^2]^{1/2}}$	
	For bolted end plate connection	
	$b_{eff,c,wc} = t_{fb} + 2\sqrt{2} a_p + 5(t_{fc} + s) + s_p$	

Reference	Calculation	Output
Figure 6.6	 t_{fb} = 15.6 mm a_p = 8.4 mm t_{fc} = 20.5 mm t_p = 25.0 mm t_{wc} = 12.8 mm	
	s_p = 2 * t_p = 50.0 mm	
	For a rolled I or H section column, s = r_c = 12.7 mm	
	$b_{eff,c,wc}$ = 248.4 mm	
	A_{vc} = 3414 mm^2	
	ω = ω_1 = 0.69	
Cl.6.2.6.2 (2)	Assume longitudinal compressive stress, $\sigma_{com,Ed}$ < 0.7 $f_{y,wc}$	
	k_{wc} = 1.0	
	t_{wc} = 12.8 mm	
	$\dfrac{\omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{Mo}}$ = 842573 N	
	Column web Bearing resistance = 842.573 kN	F_7 = 842.57 kN
Eq[n] (6.13c)	$\bar{\lambda}p = 0.932 \left(\dfrac{b_{eff,c,wc}\, d_{wc}\, f_{y,wc}}{E\, t_{wc}^2}\right)^{1/2}$	
	$b_{eff,c,wc}$ = 248.4 mm	
	for rolled I or H section column: d_{wc} = h_c - 2 (t_{fc} + r_c)	
	h_c = 267 mm	
	t_{fc} = 20.5 mm	
	r_c = 12.7 mm	
	d_{wc} = 200.30 mm	
	E = 210.0 kN/mm^2	
	$\bar{\lambda}p$ = 0.58	
	$\bar{\lambda}p$ < 0.72	
	ρ = Buckling reduction factor = 1	
	$\dfrac{\omega\, k_{wc}\, \rho\, b_{eff,c,wc}\, t_{wc}\, f_{y,wc}}{\gamma_{M1}}$ = 842573 N	F8 =
	Column web Buckling resistance = 842.573 kN	842.57 kN
EN 1993-1-8 :2005 T 3.4	**9. Bolt Shear**	
	Resistance of a single bolt in shear ($F_{v, Rd}$) is given by:	
	$F_{v, Rd} = \dfrac{\alpha_v\, f_{ub}\, A}{\gamma_{M2}}$	
	Where; α_v = 0.6 for class 8.8 bolts	
	A = A_s = 353 mm^2	
	$F_{v, Rd} = \dfrac{0.6 \times 800 \times 353}{1.25} \times 10^{-3}$ = 135.552 kN	
	V_{Rd} = n $F_{v, Rd}$	
	No: of Bolts in Shear = 6	
	Shear Resistance of the connection = 813.312 kN	F_9 = 813.312 kN

Reference	Calculation	Output
EN 1993-1-8 :2005 T 3.4	**10. Bolt Bearing** The bearing Resistance of a single bolt ($F_{b, Rd}$) is given by: $F_{b, Rd} = \frac{k_1 \alpha_b f_{ub} d t_p}{\gamma_{M2}}$ Where α_b is the least value of α_d, $\frac{f_{ub}}{f_{y,p}}$ and 1  For the Direction of load transfer For end Bolts $\alpha_d = \frac{e_1}{3d_0} = \frac{25}{3x \quad 26} = 0.32$ For inner Bolts $\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4} = \frac{60}{3x \quad 26} - \frac{1}{4} = 0.52$ $\frac{f_{ub}}{f_{u,p}} = \frac{800}{410} = 1.95$ $\alpha_b = 0.32$ For the perpendicular to the Direction of load transfer For edge bolts k1, is the smaller of $2.8 \frac{e_2}{d_0} - 1.7$ or 2.5 $2.8 \frac{e_2}{d_0} - 1.7 = 2.8 x \frac{75}{26} - 1.7 = 7.85$ Therefore for edge bolts, $k_1 = 2.50$ For inner bolts k1, is the smaller of $1.4 \frac{p_2}{d_0} - 1.7$ or 2.5 $1.4 \frac{p_2}{d_0} - 1.7 = 1.4 x \frac{100}{26} - 1.7 = 3.68$ Therefore for inner bolts, $k_1 = 2.50$ Therefore the minimum bearing resistance for a bolt is: $F_{b, Rd} = \frac{2.50 \ x \ 0.32 \ x \ 410 \ x \ 24 \ x \ 25}{1.10}$ $= 179.196$ kN bearing resistance of the connection = 6 * 179.196 = 1075.17 kN	 F_{10} = 1075.17 kN
Cl.6.2.6.7 Eq^n (6.21)	**11. Beam flange and web in compression** $F_{c,fb,Rd} = M_{c,Rd} / (h - t_{fb})$ $M_{c,Rd}$ = Design resistance of the beam assume that the design shear force in the beam doesn't reduce $M_{c,Rd}$ therefore, from P363 $M_{c,Rd}$ = 649 kNm $F_{c,fb,Rd} = \frac{649}{533 - 15.6} = 1254.11$ kN	 F_{11} = 1254.11 kN

Reference	Calculation	Output
	Summary of tension resistance	

	Column flange bending	Column web in tension	End plate in bending	beam web in tension	minimum	effective resistance
Row 1, alone	398.36	712	406.66	N/A	398.36	398.36
Row 2,alone	398.36	712	406.66	672.42	398.36	
Row 2, with row 1	690.89		813.31	N/A	690.89	
Row 2					292.53	292.53

Column web in Transverse compression = 842.57 kN

Beam flange and web in compression is not critical

<u>Moment resistance</u>

Effective resistance of bolt rows

The effective resistance of each of the three bolt rows in tension zone

$F_{t1,Rd}$ = 398.36 kN

$F_{t2,Rd}$ = 292.53 kN

Effective resistance should be reduced if the resistance of one of the higher rows exceeds

1.9 x F t,Rd = 386.323 kN

$F_{t1,Rd}$ = 386.32 kN

Total effective tension resistance

$\sum$ F t,Rd = 386.32 + 292.53

= 678.85 kN

Compression resistance = 842.57 kN

Moment resistance of the beam to column joint

= 487.2 x 386.32 + 397.2 x 292.53

= 304 kNm

Reference	Calculation	Output
	M_p = Design plastic moment resistance of beam	
	$M_p = (p_y . Z)/\gamma_{m0}$	
	$= \dfrac{275 \times 2072 \times 1000}{1.0}$	
EN1993-1-8 :2005 CL5.2.3	= 569.8 kNm	
	$M_{con}/M_{p,\,beam}$ = 0.53424	
	Hence semi rigid connection	

Reference	Calculation	Output
Table 6.10 Table 6.11 BS EN1993-1-8 :2005	**Determination of rotational stiffness for EEP 3d** **Stiffness coefficient** **1.Column web panel in shear** $k1 = (0.38\ A_{vc})/\beta Z$ Z = Lever arm β = Transformation parameter A_{vc} = Shear area of the column $k1 = (0.38\ X\ (266.7x12.8))/(1X\ (605-25-60-25-15.6/2))$ = 2.66 **2.Column web in compression** $k2 = (0.7\ b_{eff,c,wc}\ X\ t_{wc})/d_c$ $b_{eff,c,wc}$ = effective width t_{wc} = column web thickness d_c = clear depth of column $k2 = (0.7\ X 248.4\ X 12)/200.3$ k2 = 11.11 **3. Column web in tension** $k_{3,1} = (0.7\ X b_{eff,t1wc}\ X\ t_{wc})/d_c$ $k_{3,1} = (0.7\ X\ 161.5\ X\ 12.8)/200.3$ $k_{3,1}$ = 7.22 $k_{3,2} = (0.7\ X\ 161.5\ X\ 12.8)/200.3$ $k_{3,2}$ = 7.22 **4. Column flange in bending** $k_{4,1} = (0.9\ X\ l_{eff}\ X\ t_{fc}^{3})/m^3$ $k_{4,1} = (0.9\ X\ 161.5\ X 20.5^3)/33.44^3$ $k_{4,1}$ = 33.49	

Reference	Calculation	Output
	$k_{4,2}=(0.9\ X\ 161.5\ X 20.5^3\ )/33.44^3$	
	$k_{4,2}$ = 33.49	
	5. End plate bending	
	$k_{5,1}=(0.9\ X\ l_{eff}\ X\ \ t_p{}^3\ )/m^3$	
	$k_{5,1}=(0.9\ X\ 210.15\ \ X\ 25^3\ )/38.55^3$	
	$k_{5,1}$ = 51.58	
	$k_{5,2}=(0.9\ X\ 169\ \ X\ 25^3\ )/38.55^3$	
	$k_{5,2}$ = 41.48	
	6. Bolts in tension	
	$k_{10}=(1.6\ A_S\ )/L_b$	
	L_b - Bolt elongation length	
	L_b = 48.5	
	k_{10} = 11.65	
	The effective stiffness coefficient of each bolt row is obtained as follows.	
BS EN1993-1-8 :2005 CL 6.3.3.1	$k_{eff,1}=(1\ /((1/k_{3,1})+(1/k_{4,1})+(1/k_{5,1})+(1/k_{10}))$	
	$k_{eff,1}=(1\ )/((1/7.45)+(1/34.52)+(1/62.57)+(1/11.65))$	
	= 3.656	
	$k_{eff,2}=(1\ )/((1/4.25)+(1/19.7)+(1/51.58)+(1/11.65))$	
	= 3.594	
	$h_1=605-25-15.6/2-25\text{-}60$	
	h_1 = 487.2 mm	
	$h_2=605-25-15.6/2-c-(d+e)$	

Reference	Calculation	Output
	h_2 = 397.2 mm	
	Equivalent lever arm Zeq is	
BS EN1993-1-8 :2005 CL 6.3.3.1	Z_{eq} = 447.16 mm	
	k_{eq} = 7.18	
BS EN1993-1-8 :2005 CL 6.3.1	**The initial joint stiffness**	
	$S_{j,ini}=(E\,Z^{2})/(\mu\sum(1/ki))$	
	μ = 1	
	$S_{j,ini}=(210x10^3 x506.24^2)/((1/9.65)+(1/2.41)+(1/11.11))$	
	= 6.94E+07 kNmm / rad	
BS EN1993-1-8 :2005 CL5.2.2.5 figure 5.4	$EI_{b/}L_b=210x10^3x55227x10^4/6000x10^3$	
	= 1.93E+07 kNmm / rad	
	I_b - second moment of area	
	L_b - span of beam	
	$0.5E * I_b/L_b$ < $S_{j,ini}$ < $8*E*I_b/L_b$	
	$S_{j,ini}/EI_{b/}L_b$= 3.59	
	Hence semi rigid connection	

L	EIb/Lb	$S_{j,ini}/EI_b/L_b$	Classification
4000	28994175	2.39	semi rigid
6000	19329450	3.59	semi rigid
8000	14497087.5	4.79	semi rigid
10000	11597670	5.99	semi rigid
12000	9664725	7.18	semi rigid