Current Date: 20/07/2021 11:36 AM
Units system: SI
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Steel connections

Results

Connection name : BEP_AS_BCF
Connection ID : 1

Family: Beam - Column flange (BCF)
Type: Bolted end plate
Design code: AS4100-1998

DEMANDS

Description	Beam						
	N*ftop [kN]	N*fbot [kN]	N*w [kN]	M*w [kN]	V*v [kN]	N*fr [kN]	V*c [kN]
DL	168.38	-168.38	0.00	0.00	47.00	0.00	-147.38

GEOMETRIC CONSIDERATIONS

Dimensions	Unit	Value	Min. value	Max. value	Sta.	References
CHECK 1 - Detailing requirements						
End plate width, bi	[mm]	140.00	170.00	145.00	❌	DG12 Sec. 9.1
$b_{imin} = b_{fb} + 20$ [mm]						
= 150[mm] + 20 [mm]						
= **170**[mm]						DG12 Sec. 9.1
						ASI DG12 Table 3
						ASI DG12 Table 3
$b_{imax} = b_{fc} + 20$ [mm]						
= 125[mm] + 20 [mm]						
= **145**[mm]						DG12 Sec. 9.1
						ASI DG12 Table 3
						ASI DG12 Table 3
Bolt gauge, sg	[mm]	70.00	120.00	75.00	❌	DG12 Sec. 9.1
s_{gmin} = 120 [mm]						DG12 Sec. 9.1
						ASI DG12 Table 3
						ASI DG12 Table 3
$s_{gmax} = Min(b_{fb}, b_{fc} - 2.5*d_f)$						
= Min(150[mm], 125[mm] - 2.5*20[mm])						
= **75**[mm]						DG12 Sec. 9.1
						ASI DG12 Table 3

						ASI DG12 Table 3
Edge distance, ae	[mm]	35.00	30.00	50.00	✔	DG12 Sec. 9.1
a_{emin} = 30 [mm]						DG12 Sec. 9.1
a_{emax} = 2.5*d_f						
= 2.5*20[mm]						
= **50**[mm]						DG12 Sec. 9.1
End plate thickness, ti	[mm]	16.00	16.00	40.00	✔	ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
Bolt diameter, df	[mm]	20.00	20.00	610.00	✔	ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
Beam size (unhaunched)	[mm]	300.00	200.00	610.00	✔	ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
Clearence, top spo	[mm]	65.00	40.00	75.00	✔	ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3
Clearence, bottom spo	[mm]	65.00	40.00	75.00	✔	ASI DG12 Table 3
						ASI DG12 Table 3
						ASI DG12 Table 3

<u>CHECK 16 - Flange doubler stiffener</u>

Plate width, bsd	[mm]	53.50	59.50	53.50	✘	DG. 12 p. 51
b_{sd} = (b_{fb} - (t_{wc} + 2.0*r_c))/2.0						
= (125[mm] - (6[mm] + 2.0*0[mm]))/2.0						
= **59.5**[mm]						DG. 12 p. 51
b_{sd} = (b_{fc} - (t_{wc} + 2.0*r_c) - 2.0*t_w)/2.0						
= (125[mm] - (6[mm] + 2.0*0[mm]) - 2.0*6[mm])/2.0						
= **53.5**[mm]						DG. 12 p. 51

<u>CHECK 22 - Transverse stiffeners</u>						
Plate width, bs	[mm]	54.00	47.00	59.50	✔	DG. 12 p. 62

$b_{sb} = (b_{fb} - t_{wb})/2.0$
$= (150[mm] - 6.5[mm])/2.0$
$= \mathbf{71.75}[mm]$ — DG. 12 p. 62

$b_{sc} = (b_{fb}/3.0) - (t_{wc}/2.0)$
$= (150[mm]/3.0) - (6[mm]/2.0)$
$= \mathbf{47}[mm]$ — DG. 12 p. 62

$b_{smax} = (b_{fc} - t_{wc})/2.0$
$= (125[mm] - 6[mm])/2.0$
$= \mathbf{59.5}[mm]$ — DG. 12 p. 62

Plate length, ds	[mm]	212.00	97.20	232.00	✔	DG. 12 p. 62

$d_s = 1.80*b_s$
$= 1.80*54[mm]$
$= \mathbf{97.2}[mm]$ — DG. 12 p. 62

$d_s = 0.50*d_c$
$= 0.50*250[mm]$
$= \mathbf{125}[mm]$ — DG. 12 p. 62

$d_{smin} = Min(115 [mm], d_s, d_s)$
$= Min(115 [mm], 97.2[mm], 125[mm])$
$= \mathbf{97.2}[mm]$ — DG. 12 p. 62

Plate thickness, ts	[mm]	10.00	4.50	--	✔	DG. 12 p. 62

$t_{smin} = 0.5*t_{fb}$
$= 0.5*9[mm]$
$= \mathbf{4.5}[mm]$ — DG. 12 p. 62

DESIGN CHECK

Verification	**Unit**	**Capacity**	**Demand**	**Ctrl EQ**	**Ratio**	**References**
<u>CHECK 2 - Capacity of flange welds to beam</u>						
- Full penetration butt welds - no design check necessary						
<u>CHECK 3 - Capacity of web welds to beam</u>						
Web fillet weld shear capacity	[KN]	275.65	47.00	DL	0.17	Handbook 1 Sec. 4.5, Cl 9.7.3.10

$\phi v_w = \phi*0.6*f_{uw}*0.7071*t_w*k_r$
$= 0.8*0.6*480[N/mm2]*0.7071*6[mm]*1$
$= \mathbf{0.977}[kN/mm]$ — Handbook 1 Sec. 4.5

$\phi V_w = 2 * (\phi v_w*l_w)$
$= 2 * (0.977[kN/mm]*141[mm])$
$= \mathbf{275.654}[kN]$ — Cl 9.7.3.10

Web fillet weld axial capacity	[KN]	275.65	230.96	DL	0.84	Handbook 1 Sec. 4.5, Cl 9.7.3.10

$\phi v_w = \phi*0.6*f_{uw}*0.7071*t_w*k_r$
$= 0.8*0.6*480[N/mm2]*0.7071*6[mm]*1$
$= \mathbf{0.977}[kN/mm]$ — Handbook 1 Sec. 4.5

$\phi V_w = 2 * (\phi v_w * l_w)$
= 2 * (0.977[kN/mm]*141[mm])
= **275.654**[kN] Cl 9.7.3.10

$\phi N_{wt} = 0.9*f_{yw}*t_{wb}*L_{wt}$
= 0.9*280[N/mm2]*6.5[mm]*141[mm]
= **230.958**[kN] DG12 Sec. 9.3

CHECK 4 - Capacity of bolts at tension flange

Top bolts moment capacity	[KN*m]	73.63	49.00	DL	**0.67**	Cl. 9.3.2.2, DG12 Sec. 9.4

$\phi N_{tf} = \phi*A_s*f_{uf}$
= 0.8*244.794[mm2]*830[N/mm2]
= **162.543**[kN] Cl. 9.3.2.2

$\phi M_{bt} = 2*\phi N_{tf}*\Sigma d_i$
= 2*162.543[kN]*226.5[mm]
= **73.632**[kN*m] DG12 Sec. 9.4

$M*_{axial} = Min(N*_{fr}*(d_b - t_{fb}), 0.25*M*)$
= Min(0[kN]*(300[mm] - 9[mm]), 0.25*49[kN*m])
= **0**[kN*m] DG12 Sec. 9.4

$M*_{eq} = M* + M*_{axial}$
= 49[kN*m] + 0[kN*m]
= **49**[kN*m] DG12 Sec. 9.4

CHECK 5 - Capacity of bolts in shear

Bolts shear capacity	[KN]	185.24	47.00	DL	**0.25**	Cl. 9.3.2.1, DG12 Sec. 9.5

$\phi V_{bi} = Min(0.9*3.2*d_f*t_i*f_{ui}, 0.9*a_{ey}*t_i*f_{ui})$
= Min(0.9*3.2*20[mm]*16[mm]*410[N/mm2], 0.9*104[mm]*16[mm]*410[N/mm2])
= **377.856**[kN] DG12 Sec. 9.5

$\phi V_{bc} = Min(0.9*3.2*d_f*t_{fc}*f_{uc}, 0.9*a_{ey}*t_{fc}*f_{uc})$
= Min(0.9*3.2*20[mm]*9[mm]*410[N/mm2], 0.9*5069[mm]*9[mm]*410[N/mm2])
= **212.544**[kN] DG12 Sec. 9.5

$k_r = 1.0$ Cl. 9.3.2.1

$\phi V_{fn} = \phi*0.62*f_{uf}*k_r*(n_n*A_c)$
= 0.8*0.62*830[N/mm2]*1*(1*224.982[mm2])
= **92.62**[kN] Cl. 9.3.2.1

$\phi V_{df} = Min(Min(\phi V_{fn}, \phi V_{bi}), \phi V_{bc})$
= Min(Min(92.62[kN], 377.856[kN]), 212.544[kN])
= **92.62**[kN] DG12 Sec. 9.5

$\phi V_{fb} = n_{cw}*(\phi V_{df})$
= 2*(92.62[kN])
= **185.241**[kN] DG12 Sec. 9.5

CHECK 6 - Capacity of end plate at tension flange

End plate tension top flange capacity	[KN*m]	72.47	49.00	DL	0.68	DG12 Sec. 9.6, DG12 Sec. 9.4

$a_h = 0.5*(b_i*s_g)^{1/2}$
$= 0.5*(140[mm]*70[mm])^{1/2}$
$=$ **49.497**[mm] DG12 Sec. 9.6

$Y_p = 0.5*b_i*(d_{11}*(1/s_{pi} + 1/a_h) - 0.5) + 2/s_g*(d_{11}*(s_{pi} + a_h))$
= 0.5*140[mm]*(226.5[mm]*(1/60[mm] + 1/49.497[mm]) - 0.5) + 2/70[mm]*(226.5[mm]*(60[mm] + 49.497[mm]))
= **1258.17**[mm] DG12 Sec. 9.6

$\phi M_{pt} = 0.9*f_{yi}*t_i^2*Y_p$
$= 0.9*250[N/mm2]*16[mm]^2*1258.17[mm]$
= **72.471**[kN*m] DG12 Sec. 9.6

$M^*_{axial} = Min(N^*_{fr}*(d_b - t_{fb}), 0.25*M^*)$
= Min(0[kN]*(300[mm] - 9[mm]), 0.25*49[kN*m])
= **0**[kN*m] DG12 Sec. 9.4

$M^*_{eq} = M^* + M^*_{axial}$
= 49[kN*m] + 0[kN*m]
= **49**[kN*m] DG12 Sec. 9.4

CHECK 7 - Capacity of end plate in shear

- No bolts outside tension flange - check not required

CHECK 12 - Local yielding of column web at beam compression flange

Column web yielding at beam bottom flange	[KN]	253.26	168.38	DL	0.66	DG. 12 p. 65, DG. 11 p. 22, DG. 12 p. 45

$b_{sc} = t_{fb}$
= **9**[mm] DG. 12 p. 65

$c_t = 1$ DG. 11 p. 22

$\phi R_{wy} = \phi*(b_{sc} + 5.0*t_i*c_t + 5.0*t_{fc}*c_t)*t_{wc}*1.25*f_{ycw}$
= 0.9*(9[mm] + 5.0*16[mm]*1 + 5.0*9[mm]*1)*6[mm]*1.25*280[N/mm2]
= **253.26**[kN] DG. 12 p. 45

CHECK 13 - Column web crippling at beam compression flange

Column web crippling at beam bottom flange	[KN]	209.61	168.38	DL	0.80	DG. 12 p. 65, DG. 11 p. 26

$b_{sc} = t_{fb}$
= **9**[mm] DG. 12 p. 65

$\phi R_{wc} = (\phi*0.8*t_{wc}^2)*(1 + 3*(b_{sc}/d_c)*(t_{wc}/t_{fc})^{1.5})*((200000 [MPa]*f_{ycw}*t_{fc})/t_{wc})^{1/2}$
$= (0.75*0.8*6[mm]^2)*(1 + 3*(9[mm]/250[mm])*(6[mm]/9[mm])^{1.5})*((200000 [MPa]*280[N/mm2]*9[mm])/6[mm])^{1/2}$
= **209.605**[kN] DG. 11 p. 26

CHECK 14 - Column web compression buckling

Column web compression buckling	[KN]	261.37	168.38	DL	0.64	DG. 11 p. 28

$b_{sc} = t_{fb}$
= **9**[mm] DG. 12 p. 65

$b_{bc} = b_{sc} + 5.0*t_i + 5.0*t_{fc} + d_5$
= 9[mm] + 5.0*16[mm] + 5.0*9[mm] + 232[mm]
= **366**[mm] DG. 12 p. 49

$A_{wc} = b_{bc}*t_{wc}$
$= 366$[mm]$*6$[mm]
$= \mathbf{2196}$[mm2] DG. 11 p. 28

$\lambda_n = (2.5*(d_c - k_c top - k_c bottom)/t_{wc})*(k_f)^{1/2}*(f_{ywc}/(250 \text{ [MPa]}))^{1/2}$
$= (2.5*(250$[mm] $- 9$[mm] $- 9$[mm]$)/6$[mm]$)*(1)^{1/2}*(280$[N/mm2]$/(250$ [MPa]$))^{1/2}$
$= \mathbf{102.302}$ DG. 11 p. 28

$\alpha_a = 2100*(\lambda_n - 13.5)/(\lambda_n^2 - 15.3*\lambda_n + 2050)$
$= 2100*(102.302 - 13.5)/(102.302^2 - 15.3*102.302 + 2050)$
$= \mathbf{17.03}$ DG. 12 p. 49

$\alpha_b = 0.50$ DG. 12 p. 49

$\lambda = \lambda_n + \alpha_a*\alpha_b$
$= 102.302 + 17.03*0.5$
$= \mathbf{110.817}$ DG. 12 p. 49

$\eta = \mathrm{Max}(0.00326*(\lambda - 13.5), 0)$
$= \mathrm{Max}(0.00326*(110.817 - 13.5), 0)$
$= \mathbf{0.317}$ Cl. 6.3.3

$\varepsilon = ((\lambda/90)^2 + 1 + \eta)/(2.0*(\lambda/90)^2)$
$= ((110.817/90)^2 + 1 + 0.317)/(2.0*(110.817/90)^2)$
$= \mathbf{0.934}$ DG. 12 p. 49

$\alpha_c = \varepsilon*(1 - (1 - (90/(\varepsilon*\lambda))^2)^{1/2})$
$= 0.934*(1 - (1 - (90/(0.934*110.817))^2)^{1/2})$
$= \mathbf{0.472}$ DG. 12 p. 49

$\phi R_{fcb} = 0.90*\alpha_c*k_f*A_{wc}*f_{ywc}$
$= 0.90*0.472*1*2196$[mm2]$*280$[N/mm2]
$= \mathbf{261.366}$[kN] DG. 11 p. 28

CHECK 15 - Column web panel in shear

Column web panel in shear resistance	[KN]	210.47	147.38	DL	0.70	DG. 11 p. 29

$\phi N_s = 0.9*f_{yc}*A_n*k_f$
$= 0.9*260$[N/mm2]$*3697$[mm2]$*1$
$= \mathbf{865.098}$[kN] DG. 11 p. 29

$k_N = 1$ DG. 11 p. 29

$A_{wc} = (d_c - t_{fctop} - t_{fcbot})*t_{wc}$
$= (250$[mm] $- 9$[mm] $- 9$[mm]$)*6$[mm]
$= \mathbf{1392}$[mm2] DG. 11 p. 29

$\phi V_c = \phi*(0.6*f_{ywc}*A_{wc})*k_N$
$= 0.9*(0.6*280$[N/mm2]$*1392$[mm2]$)*1$
$= \mathbf{210.47}$[kN] DG. 11 p. 29

CHECK 22 - Column with transverse stiffeners at tension flange

Column flange local bending at beam top flange	[KN]	23.63	81.73	DL	3.46	Cl. 9.3.2.2, DG12 Sec. 9.4

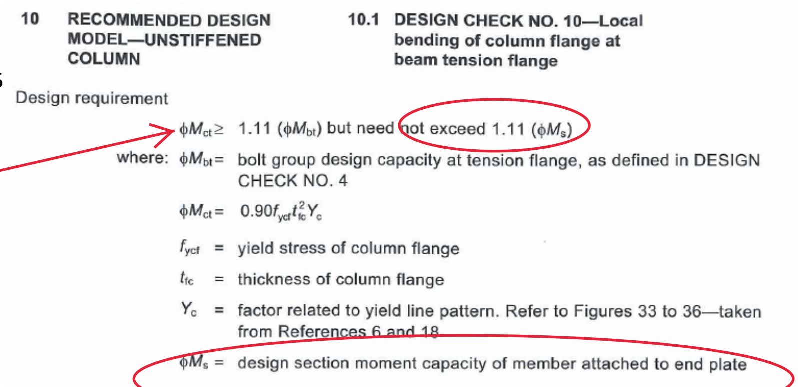
$a_h = 0.5*(b_i*s_g)^{1/2}$
$= 0.5*(140$[mm]$*70$[mm]$)^{1/2}$
$= \mathbf{49.497}$[mm] DG12 Sec. 9.6

$Y_{cs} = 0.5*b_{fc}*((d_{11}/a_h) + (d_{11}/s_{pi})) + 2.0/s_g*(d_{11}*(a_h + s_{pi}))$
= 0.5*125[mm]*((226.5[mm]/49.497[mm]) + (226.5[mm]/60[mm])) + 2.0/70[mm]*(226.5[mm]*(49.497[mm] + 60[mm]))
= **1230.54**[mm] DG. 12 p. 63

$\phi M_{ctd} = \phi*(f_{ycf}*t_{fc}^2 + f_{yd}*t_d^2)*Y_c$
= 0.9*(260[N/mm2]*9[mm]^2 + 280[N/mm2]*1[mm]^2)*1230.54[mm]
= **23.634**[kN*m] DG. 12 p. 51

$\phi N_{tf} = \phi*A_s*f_{uf}$
= 0.8*244.794[mm2]*830[N/mm2]
= **162.543**[kN] Cl. 9.3.2.2

$\phi M_{bt} = 2*\phi N_{tf}*\Sigma d_i$
= 2*162.543[kN]*226.5
= **73.632**[kN*m] DG12 Sec. 9.4


10 RECOMMENDED DESIGN MODEL—UNSTIFFENED COLUMN

10.1 DESIGN CHECK NO. 10—Local bending of column flange at beam tension flange

Design requirement

$\phi M_{ct} \geq$ 1.11 (ϕM_{bt}) but need not exceed 1.11 (ϕM_s)

where: ϕM_{bt} = bolt group design capacity at tension flange, as defined in DESIGN CHECK NO. 4

ϕM_{ct} = $0.90 f_{ycf} t_{fc}^2 Y_c$

f_{ycf} = yield stress of column flange

t_{fc} = thickness of column flange

Y_c = factor related to yield line pattern. Refer to Figures 33 to 36—taken from References 6 and 18

ϕM_s = design section moment capacity of member attached to end plate

$\phi M_{cts} = 1.11*\phi M_{bt}$
= 1.11*73.632[kN*m]
= **81.732**[kN*m] DG. 12 p. 61

Top flange yield capacity DG. 12 p. 61, DG. 12 p. 41

$A_{sn} = 2.0*b_s*t_s$
= 2.0*54[mm]*10[mm]
= **1080**[mm2] DG. 12 p. 61

$\phi R_{fts} = 0.90*f_{ys}*A_s$
= 0.90*260[N/mm2]*1080[mm2]
= **252.72**[kN] DG. 12 p. 61

$a_h = 0.5*(b_i*s_g)^{1/2}$
= 0.5*(140[mm]*70[mm])$^{1/2}$
= **49.497**[mm] DG12 Sec. 9.6

$Y_c = 0.5*b_{fc}*(d_{11}/a_h) + (2.0/s_g)*(d_{11}*(a_h + 0.75*s_{pi}) + 0.5*(s_{pi}^2)) + 0.5*s_g$
= 0.5*125[mm]*(226.5[mm]/49.497[mm]) + (2.0/70[mm])*(226.5[mm]*(49.497[mm] + 0.75*60[mm]) + 0.5*(60[mm]^2)) + 0.5*70[mm]
= **983.962**[mm] DG. 12 p. 41

$\phi M_{ct} = 0.90*f_{ycf}*t_{fc}^2*Y_c$
= 0.90*260[N/mm2]*9[mm]^2*983.962[mm]
= **18.65**[kN*m] DG. 12 p. 41

$\phi R_{ft} = \phi M_{ct}/(d_b - t_{fb})$
= 18.65[kN*m]/(300[mm] - 9[mm])
= **64.089**[kN] DG. 12 p. 61

$b_{sc} = t_{fb}$
= **9**[mm] DG. 12 p. 65

$c_t = 0.5$ DG. 11 p. 22

$\phi R_{wt} = \phi*(b_{sc} + 2.0*t_i*c_t + 6.0*k_c*c_t)*t_{wc}*f_{ycw}$
= 0.9*(9[mm] + 2.0*16[mm]*0.5 + 6.0*9[mm]*0.5)*6[mm]*280[N/mm2]
= **78.624**[kN] DG. 12 p. 44

Top flange end weld capacity	[KN]	130.25	104.30	DL	0.80	Handbook 1 Sec. 4.5

$\phi v_w = \phi*0.6*f_{uw}*0.7071*t_w*k_r$
= 0.8*0.6*410[N/mm2]*0.7071*6[mm]*1
= **0.835**[kN/mm] Handbook 1 Sec. 4.5

$L_w = Min(4.0*(b_s - 15$ [mm]$), 4.0*(d_s - 15$ [mm]$))$
= Min(4.0*(54[mm] - 15 [mm]), 4.0*(212[mm] - 15 [mm]))
= **156**[mm] DG. 12 p. 61

$\phi R_{ftw} = \phi v_w*L_w$
= 0.835[kN/mm]*156[mm]
= **130.251**[kN] DG. 12 p. 61

$a_h = 0.5*(b_i*s_g)^{1/2}$
= $0.5*(140$[mm]$*70$[mm]$)^{1/2}$
= **49.497**[mm] DG12 Sec. 9.6

$Y_c = 0.5*b_{fc}*(d_{11}/a_h) + (2.0/s_g)*(d_{11}*(a_h + 0.75*s_{pi}) + 0.5*(s_{pi}^2)) + 0.5*s_g$
= 0.5*125[mm]*(226.5[mm]/49.497[mm]) + (2.0/70[mm])*(226.5[mm]*(49.497[mm] + 0.75*60[mm]) + 0.5*(60[mm]^2)) + 0.5*70[mm]
= **983.962**[mm] DG. 12 p. 41

$\phi M_{ct} = 0.90*f_{ycf}*t_{fc}^2*Y_c$
= 0.90*260[N/mm2]*9[mm]^2*983.962[mm]
= **18.65**[kN*m] DG. 12 p. 41

$\phi R_{ft} = \phi M_{ct}/(d_b - t_{fb})$
= 18.65[kN*m]/(300[mm] - 9[mm])
= **64.089**[kN] DG. 12 p. 61

$b_{sc} = t_{fb}$
= **9**[mm] DG. 12 p. 65

$c_t = 0.5$ DG. 11 p. 22

$\phi R_{wt} = \phi*(b_{sc} + 2.0*t_i*c_t + 6.0*k_c*c_t)*t_{wc}*f_{ycw}$
= 0.9*(9[mm] + 2.0*16[mm]*0.5 + 6.0*9[mm]*0.5)*6[mm]*280[N/mm2]
= **78.624**[kN] DG. 12 p. 44

Global critical strength ratio **3.46**

NOTES

- Unsuitable steel beam grade for recommended detailing, should be grade 300 or grade 350.

NOTATION

A_c:	Minor diameter area of the bolt
a_{emax}:	Maximum edge distance
a_{emin}:	Minimum edge distance
a_{ey}:	min (ae2, ae1-1)
a_h:	Tension flange external yielding line distance
A_n:	Net area of the cross section
A_s:	Tensile stress area of the bolt
A_{sn}:	Net stiffener area
A_{wc}:	Area of web resisting shear force
α_a:	Compression member factor
α_b:	Compression member section constant
α_c:	Compression member slenderness reduction factor
b_{bc}:	Distance dispersion line

b_{fb}: Beam flange width
b_{fc}: Column flange width
b_i: Plate width
b_{imax}: Maximum plate width
b_{imin}: Minimum plate width
b_s: Stiffener width
b_{sb}: Minimum stiffener width at beam side
b_{sc}: Minimum stiffener width at column side
b_{sc}: Stiff bearing dimension
b_{sd}: Doubler plate width
b_{smax}: Maximum stiffener width
c_t: Continuing column
d_{11}: Distances from centre of beam compression flange to centre of bolt row
d_5: Dispersion distance
d_b: Bolt diameter
d_c: Depth of cope
d_c: Column depth
d_f: Nominal bolt diameter
d_s: Length of stiffener
d_s: Minimum stiffener length
d_s: Minimum stiffener length
d_{smin}: Minimum stiffener length
ε: Compression member factor
η: Compression member imperfection factor
f_{uc}: Tensile strength of supporting member
f_{uf}: Minimum tensile strength of bolt
f_{ui}: Tensile strength of component
f_{uw}: Tensile strength of supported member web
f_{yc}: Yield stress of supporting column web/wall
f_{ycf}: Yield stress of column flange
f_{ycw}: Yield stress of column web
f_{yd}: Yield stress of any flange doubler plate present
f_{yi}: Yield strength of component
f_{ys}: Yield stress of the stiffener
f_{yw}: Beam web yield stress
f_{ywc}: Yield stress of column web
k_c: Distance on column section from outer face of flange to inner termination of root radius
$k_c$bottom: Distance on column section from bottom outer face of flange to inner termination of root radius
$k_c$top: Distance on column section from top outer face of flange to inner termination of root radius
k_f: Section form factor
k_N: Factor
k_r: Reduction factor for lap splice connections
l_w: Fillet lenght
L_w: Total weld group fillet length
L_{wt}: Distance from radius to inside face of tension flange to mid-depth of beam along end plate
λ: Slenderness
λ_n: Modified compression member slenderness
M^*: Design bending moment at connection
M^*_{axial}: Equivalent design moment
M^*_{eq}: Equivalent design moment
n_{cw}: Number of bolts in compression
n_n: Number of shear planes with threads intercepting the shear plane
N^*_{fr}: Flange force due to tension/shear
ϕ: Capacity factor
ϕM_{bt}: Design capacity of beam section at tension
ϕM_{ct}: Design capacity of column section at tension
ϕM_{ctd}: Design capacity of column flange section at tension
ϕM_{cts}: Design capacity of column section at tension
ϕN_s: Design section capacity in compression
ϕN_{tf}: Design tensile capacity for bolt
ϕR_{fcb}: Design buckling capacity of stiffener and web acting together
ϕR_{ft}: Tension flange capacity
ϕR_{fts}: Tension stiffener capacity
ϕR_{ftw}: Tension flange capacity at web
ϕR_{wc}: Column web crippling at beam compression flange capacity
ϕR_{wt}: Tension web capacity
ϕR_{wy}: Yielding of column web at beam compression flange

ϕV_c: Shear in plate capacity
ϕV_{fn}: Design capacity in shear for bolt with threads included in the shear plane
ϕv_w: Design capacity of fillet weld per unit length
ϕV_w: Design capacity of fillet weld
ϕM_{bt}: Capacity of bolts at tension flange
ϕM_{pt}: Design capacity of end plate at tension flange
ϕN_{tf}: Design capacity of bolt in tension
ϕN_{wt}: Beam web axial capacity
ϕV_{bc}: Design capacity related to local bearing or end plate tearout in the supporting column flange
ϕV_{bi}: Design capacity related to local bearing or end plate tearout in the end plate component (single bolt)
ϕV_{df}: Design capacity of a single bolt in shear for the strength limit state
ϕV_{fb}: Design capacity of bolts in shear
ϕV_{fn}: Design capacity for a single bolt threads included
r_c: Column root radius
s_g: Bolt gauge (horizontal spacing between two columns)
s_{gmax}: Maximum bolt gauge
s_{gmin}: Minimum bolt gauge
s_{pi}: Internal distance from bolt centre-line to face of flange at tension flange
Σd_i: Sum of bolt lever arms
t_d: Thickness of any flange doubler plate present
t_{fb}: Beam flange thickness
t_{fc}: Thickness of the column flange
t_{fcbot}: Column bottom flange thickness
t_{fctop}: Column top flange thickness
t_i: Connector thickness
t_s: Thickness of stiffener
t_{smin}: Minimum stiffener thickness
t_w: Weld fillet weld size
t_w: Side flange doubler weld size
t_{wb}: Beam web thickness
t_{wc}: Supporting column web/wall thickness
Y_c: Unstiffened yielding line pattern factor
Y_{cs}: Yielding line pattern factor
Y_p: Factor related to yield line pattern