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Anchor Bolt Design per ACI 318-11

Crane beam design

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This is a comparison between the combination of different size of column and beam to see how effective it is to increase the MC capacity for the following options

1. Increase beam depth
2. Increase column web & flange thickness

The following is the result, and apparently, increasing the beam depth is the more effective approach

Beam Size	Column Size	Mr kN-m
W310X60	W310X97	155
W530X66	W310X97	268
W310X60	W310X143	227

MC design is per AISC design guide 4 and 13 and the following parts will dictate if stiffener plate is required, with the reference section of AISC design guide 4 and 13 on the right

CivilBay www.civilbay.com Calculation Sheet			
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Job No :		Chk :	Test
Doc No :		Date :	2013-02-04
Subject :	www.civilbay.com	Rev :	0

Code Reference

	$P_{t\text{-}pry} = T_{avail}$	$= 251.0$	[kN]		
Moment resistance by bolt tension	$M_r = 2 \times P_{t\text{-}pry} \times (h_0 + h_1)$	$= 291.1$	[kNm]		
	ratio $= 0.54$	$> M_f$	**OK**		
End Plate Shear Yielding Subject To Flange Tension Force					*AISC Design Guide 4*
Factored flange force for design	$P_{uf} = M_f / (d_b - t_{fb})$	$=$ **547.2**	[kN]		Page 21 - item 8
Shear yielding resistance	$= \phi\, 0.6\, F_{yp}\, b_p\, t_p$	$= 1480.0$	[kN]		
	ratio $= 0.18$	$> 0.5P_{uf}$	**OK**		

Column — **Code Reference**

Column Flange Flexural Yeilding — *AISC Design Guide 4*

$s = 0.5 \times \text{sqrt}(b_{fc} \times g)$ = 99.6 [mm]

$c = p_{fo} + t_{fb} + p_{fi}$ = 113.1 [mm]

$$Y_c = \frac{b_{fc}}{2}\left(\frac{h_1}{s}+\frac{h_0}{s}\right)+\frac{2}{g}\left[h_1\left(s+\frac{3c}{4}\right)+h_0\left(s+\frac{c}{4}\right)+\frac{c^2}{2}\right]+\frac{g}{2}$$

= 2394.8 [mm] — Table 3-4

$$t_{p\ reqd} = \sqrt{\frac{1.11\ M_f}{\phi\ F_{yc} Y_c}}$$ = 15.4 [mm] — Page 21 (3.10)

ratio = 1.00 < t_{fc} **OK**

column stiffeners not required

Column Web Yeilding - Tension & Compression

MC is NOT at top of column — C_t = 1.0 — ϕ = 1.0 — Page 22 - item 16

Bearing length — $N = t_{fb} + 2 \times w_1$ = 33.1 [mm]

Column web yeilding resistance — $= \phi C_t(6k_c + N + 2t_p) F_{yc} t_{wc}$ = 1011 [kN]

ratio = 0.54 > P_{uf} **OK**

column stiffeners not required

Column Web Buckling - Compression

MC is NOT at top of column — C_t = 1.0 — ϕ = 0.9 — Page 22 - item 17

$h = d_c - 2 \times k_c$ = 247.0 [mm]

Column web buckling resistance — $= \phi C_t \frac{24 t_{wc}^3 \sqrt{E F_{yc}}}{h}$ = 705 [kN]

ratio = 0.78 > P_{uf} **OK**

column stiffeners not required

Code Reference

Case 2: beam top flange located < $0.5d_c$ from end of column

AISC Design Guide 4

N / d_c = 0.107

Page 23 - item 18

Case 2a For N/d_c <= 0.2

This case does NOT apply

Column web crippling resistance $= \phi\, 0.4 t_{wc}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$ = 0 [kN]

Case 2b For N/d_c > 0.2

This case does NOT apply

Column web crippling resistance $= \phi\, 0.4 t_{wc}^2 \left[1 + \left(\frac{4N}{d_c} - 0.2\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$ = 0 [kN]

Column web crippling resistance = = **710** [kN]

ratio = 0.77 > P_{uf} **OK**

column stiffeners not required

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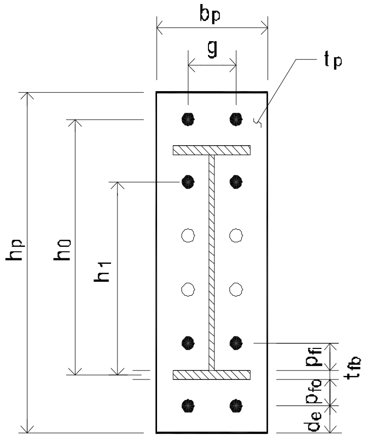
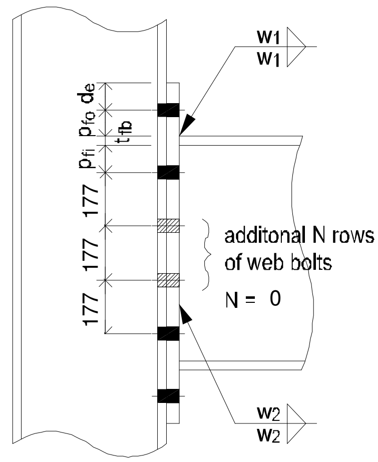
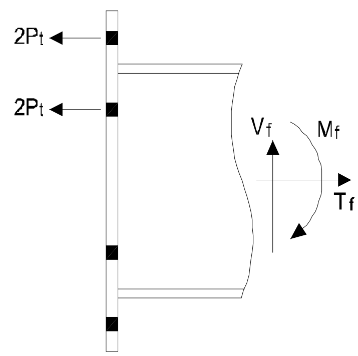
FOUR BOLT UNSTIFFENED MOMENT CONNECTION DESIGN

Four bolt unstiffened moment connection design based on	**Code Abbreviation**
CSA-S16-09 Limit States Design of Steel Structures	*CSA-S16-09*
AISC Design Guide 4: Extended End-Plate Moment Connections Seismic and Wind Applications	*AISC Design Guide 4*
AISC Design Guide 13: Wide-Flange Column Stiffening at Moment Connections	*AISC Design Guide 13*
AISC Steel Construction Manual 13th Edition	*AISC SCM 13th Ed*







INPUT **Code Reference**

Bolt grade		A325	-N	Bolt diameter d_b =	1	[in]	*CSA-S16-09*
Bolt min tensile strength	F_{ub} =	825	[MPa]				13.12.1.2
Bolt hole is		Punched	Use Punched unless it's confirmed to be Drilled				
Bolt thread is		Included	Use Included unless it's confirmed to be Excluded				
Beam properties		W_310		W310x60			
	d_b =	303	[mm]	b_{fb} =	203	[mm]	
	t_{fb} =	13.1	[mm]	t_{wb} =	7.5	[mm]	
	k_b =	25.9	[mm]				
Column properties		W_310		W310x97	b_{fc} >= b_p	**OK**	
	d_c =	308	[mm]	b_{fc} =	305	[mm]	
	t_{fc} =	15.4	[mm]	t_{wc} =	9.9	[mm]	
	k_c =	30.5	[mm]	A =	12300	[mm^2]	
W shape material strength	F_y =	345	[MPa]	F_u =	450	[MPa]	
End plate material strength	F_{yp} =	300	[MPa]	F_{up} =	450	[MPa]	
				suggest			*AISC Design Guide 4*
End plate width	b_p =	228	[mm]	only b_p = (b_{fb} + 1") used for design			Page 16
Gauge	g =	130	[mm]	130			
Bolt clear dist - inner bolt	p_{fi} =	50	[mm]	50			
Bolt clear dist - outer bolt	p_{fo} =	50	[mm]	50			
Bolt edge dist	d_e =	45	[mm]	45			
End plate thickness	t_p =	40.0	[mm]	25.4 or 31.8			

Check if MC located at top of column								**Code Reference**
Dist between beam top flange & top of column <= d_c		No	MC is NOT located at top of column					
Factored beam moment	M_{con} =	155	[kNm]					
Factored beam tensile force	T_f =	25	[kN]					
Moment converted from tension	M_{ten} =	$0.5T_f \times (d_b - t_{fb})$			= 3.6	[kNm]		
Factored moment for design	$\mathbf{M_f}$ =	$M_{con} + M_{ten}$			= **159**	[kNm]		
Factored beam shear	$\mathbf{V_f}$ =	259	[kN]		= **259**	[kN]		
Factored column axial compressio	$\mathbf{C_f}$ =	0	[kN]		= **0**	[kN]		
Weld electrode	=	E49XX		X_u	= 490	[MPa]		
				suggest				
Fillet weld - beam flange	w_1 =	10	[mm]	12 mm				
Fillet weld - beam web	w_2 =	8	[mm]	8 mm				
Min. fillet weld size	=	8	[mm]	ratio	= 1.00		OK	*CSA-W59-03* Table 4.4
Min edge distance	=	44	[mm]	ratio	= 0.98		OK	*CSA-S16-09* Table 6
Resistance factor	ϕ =	0.90		ϕ_u	= 0.75			13.1 (a)
	ϕ_b =	0.80		ϕ_{br}	= 0.80			13.1 (c) (g)
	ϕ_w =	0.67						13.1 (h)

CONCLUSION

Overall	ratio	= **1.00**	OK
Bolt			
Bolt Tension	ratio	= 0.55	OK
Bolt Shear	ratio	= 0.46	OK
Bolt Bearing & Tear Out on End Plate	ratio	= 0.08	OK
Bolt Bearing & Tear Out on Column Flange	ratio	= 0.15	OK
Weld			
Beam Flange To End Plate Fillet Weld - Tension	ratio	= 0.68	OK
Beam Web To End Plate Fillet Weld - Shear	ratio	= 0.75	OK
End Plate			
End Plate Thickness by Yield Line Method	ratio	= 0.44	OK
End Plate Thickness Considering Prying Action	ratio	= 0.54	OK
End Plate Shear Yielding Subject To Flange Tension Force	ratio	= 0.18	OK
End Plate Shear Rupture Subject To Flange Tension Force	ratio	= 0.20	OK
End Plate Shear Yielding Subject To Vertical Shear	ratio	= 0.04	OK
End Plate Shear Rupture Subject To Vertical Shear	ratio	= 0.04	OK
End Plate Block Shear	ratio	= 0.07	OK
Column			
Column Flange Flexural Yeilding	ratio	= 1.00	OK
Column Web Yeilding - Tension & Compression	ratio	= 0.54	OK
Column Web Buckling - Compression	ratio	= 0.78	OK
Column Web Crippling - Compression	ratio	= 0.77	OK
Column Panel Zone Web Shear	ratio	= 0.96	OK

DESIGN CHECK — **Code Reference**

For net area calculation — *CSA-S16-09*

1. Bolt hole dia is 2mm larger than nominal bolt dia
2. For punched hole, addtional 2mm is added to bolt hole dia when it's used in net area width calc — 12.3.2

d_h = 29.4 [mm] for Punched hole

Tension bolt lever arm h_0 = 346.4 [mm] h_1 = 233.3 [mm]

Bolt

Bolt Tension

Nominal bolt area	A_b =		= 506.7	[mm^2]	
Single bolt tensile resistance	P_t =	$0.75\ \phi_b\ A_b\ F_{ub}$	= 250.8	[kN]	13.12.1.3
Moment resistance by bolt	M_r =	$2 \times P_t \times (h_0 + h_1)$	= 291	[kNm]	
	ratio =	0.55	> M_f	**OK**	

Bolt Shear — *AISC Design Guide 4*

Assume all shear taken by compression side bolts only — Page 9 2.1-4

Bolt number taking shear n_{bv} = 4 shear plane m = 1 — *CSA-S16-09*

Bolt threads are intercepted by a shear plane

A_b =	0.7 A	= **354.7**	[mm^2]	13.12.1.2 (c)
V_r =	$0.6\ \phi_b\ n_{bv}\ m\ A_b\ F_{ub}$	= 562	[kN]	13.12.1.2 (c)
ratio =	0.46	> V_f	**OK**	

Bolt Bearing & Tear Out on End Plate

Bearing strength per bolt n_{bv} = 1 bolt dia d = 25.4 [mm]

$B_r = 3\ \phi_{br}\ n_{bv}\ t_p\ d\ F_{up}$ = 1097 [kN] 13.12.1.2 (a)

Exterior bolt tear out strength per bolt

Gross shear area $A_{gv} = d_e \times t_p \times 2$ side = 3600 [mm^2]

Tear-out resistance per bolt $T_{r1} = \phi_u 0.6 A_{gv} \dfrac{F_y + F_u}{2}$ = 608 [kN] 13.11

Exterior bolt shear resistance V_{r1} = min (B_r, T_{r1}) = 608 [kN]

Interior bolt tear out strength per bolt

Gross shear area $A_{gv} = (p_{fi} + t_{fb} + p_{fo}) \times t_p \times 2$ side = 9048 [mm^2]

Tear-out resistance per bolt $T_{r2} = \phi_u 0.6 A_{gv} \dfrac{F_y + F_u}{2}$ = 1527 [kN] 13.11

Interior bolt shear resistance V_{r2} = min (B_r, T_{r2}) = 1097 [kN]

Total shear resistance $V_r = V_{r1} \times 2 + V_{r2} \times 2$ = 3410 [kN]

ratio = 0.08 > V_f **OK**

Bolt Bearing & Tear Out on Column Flange					**Code Reference**
Bearing strength per bolt	n_{bv} = 1	bolt dia d = 25.4		[mm]	*CSA-S16-09*
	$B_r = 3\ \phi_{br}\ n_{bv}\ t_{fc}\ d\ F_u$	= 422		[kN]	13.12.1.2 (a)

Assume exterior bolt edge distance on column flange is big and tear-out not governing

Exterior bolt shear resistance	$V_{r1} = B_r$	= 422	[kN]	
Interior bolt tear out strength per bolt				
Gross shear area	$A_{gv} = (p_{fi} + t_{fb} + p_{fo}) \times t_{fc} \times 2$ side	= 3483	[mm^2]	
Tear-out resistance per bolt	$T_{r2} = \phi_u 0.6 A_{gv} \dfrac{F_y + F_u}{2}$	= 623	[kN]	13.11
Interior bolt shear resistance	$V_{r2} = \min(B_r, T_{r2})$	= 422	[kN]	
Total shear resistance	$V_r = V_{r1} \times 2 + V_{r2} \times 2$	= 1690	[kN]	
	ratio = 0.15	> V_f	**OK**	

Weld

Fillet weld resistance				
Base metal resistance	$A_m = w_1 \times 1mm$	= 10.0	[mm^2]	
	$v_{rm} = 0.67\ \phi_w\ A_m\ F_{up}$	= 2.0	[kN/mm]	13.13.2.2
Weld metal resistance	$A_w = 0.707 \times w_1 \times 1mm$	= 7.1	[mm^2]	
Angle of weld axis and force	θ =	= 0		
	$v_{rw} = 0.67\ \phi_w\ A_w\ X_u\ (1 + 0.5 \sin\theta$ ^1.5)	= 1.6	[kN/mm]	13.13.2.2

Beam Flange To End Plate Fillet Weld - Tension — *AISC Design Guide 4*

For wind and low-seismic applications, the flange force used for flange weld design shall not be less than $0.6F_y A_{fb}$ — Page 38

60% of beam flange yield strength	$P_{uf1} = 0.6 \times F_y \times b_{fb} \times t_{fb}$	= 550	[kN]	
Flange force by moment	$P_{uf2} = M_f / (d_b - t_{fb})$	= 547	[kN]	
Flange force used for design	$P_{uf} = \max(P_{uf1}, P_{uf2})$	= **550**	[kN]	
Angle of weld axis and force	θ =	= 90		
Fillet weld resistance	$v_r = \min(v_{rm}, 1.5v_{rw})$	= 2.02	[kN/mm]	
	$V_r = v_r \times (2 \times b_{fb} - t_{wb})$	= 805	[kN]	
	ratio = 0.68	> P_{uf}	**OK**	

Beam Web To End Plate Fillet Weld - Shear

Angle of weld axis and force	θ =	= 0		
Fillet weld resistance	$v_r = \min(v_{rm}, v_{rw}) \times w_2 / w_1$	= 1.2	[kN/mm]	
Shear weld length	$L_1 = 0.5d_b - t_{fb}$	= 138.4	[mm]	Page 33
	$L_2 = d_b - 2t_{fb} - p_{fi} - 2d_b$	= 176.0	[mm]	
	$V_r = v_r \times \min(L_1, L_2) \times 2$ side	= 344	[kN]	
	ratio = 0.75	> V_f	**OK**	

Code Reference

End Plate *AISC Design Guide 4*

End Plate Thickness by Yield Line Method

$s = 0.5 \times \text{sqrt}(b_p \times g)$ = 86.2 [mm] Table 3-1

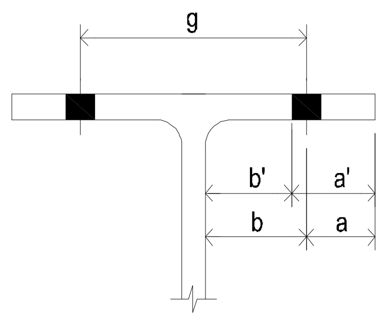
p_{fi} = s if $p_{fi} > s$ = 50.0 [mm]

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}}+\frac{1}{s}\right)+\frac{h_0}{p_{fo}}-\frac{1}{2}\right]+\frac{2}{g}\left[h_1(p_{fi}+s)\right] = 2065.3 \text{ [mm]}$$

$$t_{p\ reqd} = \sqrt{\frac{1.11\ M_f}{\phi\ F_{yp} Y}} = 17.8 \text{ [mm]}$$ Page 21 (3.10)

ratio = 0.44 < t_p **OK**

End Plate Thickness Considering Prying Action *AISC SCM 13th Ed*



a = 1.94 [in]

b = 2.41 [in]

Bolt dia d_b = 1.000 [in]

Bolt hole d' = 1.063 [in] F_u = 65 [ksi]

Bolt ver. tributary length $p = (p_{fi} + 0.5t_{fb}) \times 2$ = 4.45 [in]

$a' = (a + 0.5d_b) \leq (1.25b + 0.5d_b)$ = 2.44 [in] page 9-12

$b' = b - 0.5d_b$ = 1.91 [in] page 9-11

$\rho = b' / a'$ = 0.78

$\delta = 1 - d' / p$ = 0.761 page 9-11

End plate thickness $t = t_p$ = 1.575 [in]

From *AISC Design Guide 4* page 9 design assumption 4, all the shear force is assumed to be resisted by the compression side bolts. so there is no bolt tensile capacity reduction due to presence of shear

Tensile force per bolt without prying $B = P_t$ = 56.4 [kips]

To get full B required thickness $t_c = \sqrt{\frac{4.44\ B\ b'}{p\ F_u}}$ = 1.28 [in] page 9-12

$\alpha' = \frac{1}{\delta(1+\rho)}\left[(t_c/t)^2 - 1\right]$ = -0.25 page 9-13

Multiplier for prying action Q = = 1.000 page 9-12

Tensile force per bolt available $T_{avail} = B \times Q$ = 56.4 [kips]

Code Reference

	$P_{t\text{-}pry}$ = T_{avail}		= 251.0	[kN]	
Moment resistance by bolt tension	M_r = 2 x $P_{t\text{-}pry}$ x (h_0 + h_1)		= 291.1	[kNm]	
	ratio = 0.54		> M_f	**OK**	

End Plate Shear Yielding Subject To Flange Tension Force — *AISC Design Guide 4*

Factored flange force for design	P_{uf} = M_f / (d_b - t_{fb})		= **547.2**	[kN]	Page 21 - item 8
Shear yielding resistance	= ϕ 0.6 F_{yp} b_p t_p		= 1480.0	[kN]	
	ratio = 0.18		> $0.5P_{uf}$	**OK**	

End Plate Shear Rupture Subject To Flange Tension Force

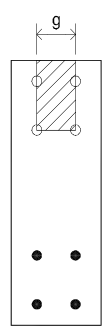
End plate net area	A_n = (b_p - 2 d_h) x t_p		= 6784	[mm^2]	Page 21 - item 9
Shear rupture resistance	= ϕ_u 0.6 F_{up} A_n		= 1373.8	[kN]	
	ratio = 0.20		> $0.5P_{uf}$	**OK**	

End Plate Shear Yielding Subject To Vertical Shear

End plate length	h_p = 493.0 [mm]	bolt hole d_h	= 29.4	[mm]	
Shear yielding resistance	= ϕ 0.6 F_{yp} h_p t_p x 2		= 6389.3	[kN]	
	ratio = 0.04		> V_f	**OK**	

End Plate Shear Rupture Subject To Vertical Shear

End plate net shear area	A_n = (h_p - 4 x d_h) x t_p x 2		= 30032	[mm^2]	
Shear rupture resistance	= ϕ_u 0.6 F_{up} A_n		= 6081.5	[kN]	
	ratio = 0.04		> V_f	**OK**	



End Plate Block Shear — *CSA-S16-09*

Net area in tension	A_n = (g- d_h) x t_p	= 4024	[mm^2]	12.3.1 (a)
Gross area in shear	A_{gv} = (p_{fi} + t_{fb} + p_{fo} + d_e) x t_p x 2 side	= 12648	[mm^2]	
	U_t = 1.0			13.11 (a)
	$V_r = \phi_u\left[U_t A_n F_u + 0.6 A_{gv}\dfrac{F_y + F_u}{2}\right]$	= 3492	[kN]	13.11
	ratio = 0.07	> V_f	**OK**	

Column — **Code Reference**

Column Flange Flexural Yeilding — *AISC Design Guide 4*

$s = 0.5 \times \text{sqrt}(b_{fc} \times g)$ = 99.6 [mm]

$c = p_{fo} + t_{fb} + p_{fi}$ = 113.1 [mm]

$$Y_c = \frac{b_{fc}}{2}\left(\frac{h_1}{s}+\frac{h_0}{s}\right)+\frac{2}{g}\left[h_1\left(s+\frac{3c}{4}\right)+h_0\left(s+\frac{c}{4}\right)+\frac{c^2}{2}\right]+\frac{g}{2}$$

= 2394.8 [mm] — Table 3-4

$$t_{p\ reqd} = \sqrt{\frac{1.11\ M_f}{\phi\ F_{yc} Y_c}}$$

= 15.4 [mm] — Page 21 (3.10)

ratio = 1.00 < t_{fc} **OK**

column stiffeners not required

Column Web Yeilding - Tension & Compression

MC is NOT at top of column C_t = 1.0 ϕ = 1.0 — Page 22 - item 16

Bearing length $N = t_{fb} + 2 \times w_1$ = 33.1 [mm]

Column web yeilding resistance $= \phi\, C_t (6k_c + N + 2t_p)\, F_{yc} t_{wc}$ = 1011 [kN]

ratio = 0.54 > P_{uf} **OK**

column stiffeners not required

Column Web Buckling - Compression

MC is NOT at top of column C_t = 1.0 ϕ = 0.9 — Page 22 - item 17

$h = d_c - 2 \times k_c$ = 247.0 [mm]

Column web buckling resistance $= \phi\, C_t \dfrac{24 t_{wc}^3 \sqrt{E\, F_{yc}}}{h}$ = 705 [kN]

ratio = 0.78 > P_{uf} **OK**

column stiffeners not required

Column Web Crippling - Compression

Bearing length $N = t_{fb} + 2 \times w_1$ = 33.1 [mm] — Page 23 - item 18

ϕ = 0.75

Case 1: beam top flange located > 0.5d_c from end of column **This case applies**

Column web crippling resistance $= \phi\ 0.8 t_{wc}^2 \left[1 + 3\left(\dfrac{N}{d_c}\right)\left(\dfrac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\dfrac{E F_{yc} t_{fc}}{t_{wc}}}$ = 710 [kN]

Code Reference

Case 2: beam top flange located < $0.5d_c$ from end of column — *AISC Design Guide 4*

$N / d_c = 0.107$ — Page 23 - item 18

Case 2a For $N/d_c <= 0.2$ — **This case does NOT apply**

Column web crippling resistance $= \phi\, 0.4 t_{wc}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$ = 0 [kN]

Case 2b For $N/d_c > 0.2$ — **This case does NOT apply**

Column web crippling resistance $= \phi\, 0.4 t_{wc}^2 \left[1 + \left(\frac{4N}{d_c} - 0.2\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$ = 0 [kN]

Column web crippling resistance = = **710** [kN]

ratio = 0.77 > P_{uf} **OK**

column stiffeners not required

Column Panel Zone Web Shear — *AISC Design Guide 13*

Column min yield strength $P_y = F_y A$ = 4244 [kN]

$C_f / P_y = 0.000$ $\phi = 0.9$

For $N_f / P_y <= 0.4$ **This case applies**

Panel zone shear resistance $R_v = \phi\, 0.6\, F_y\, d_c\, t_{wc}$ = 568 [kN] Page 6 (2.2-1)

For $N_f / P_y > 0.4$ **This case does NOT apply**

Panel zone shear resistance $R_v = \phi\, 0.6\, F_y\, d_c\, t_{wc}\,(1.4 - C_f / P_y)$ = 0 [kN] Page 6 (2.2-2)

Panel zone shear resistance R_v = = **568** [kN]

Factored flange force $P_{uf} = M_f / (d_b - t_{fb})$ = 547 [kN]

Neglect the effects of storey shear

Panel zone web shear force $V_u = P_{uf}$ = 547 [kN] Page 5 (2.1-5)

ratio = 0.96 < R_v **OK**

column web doubler plate not required

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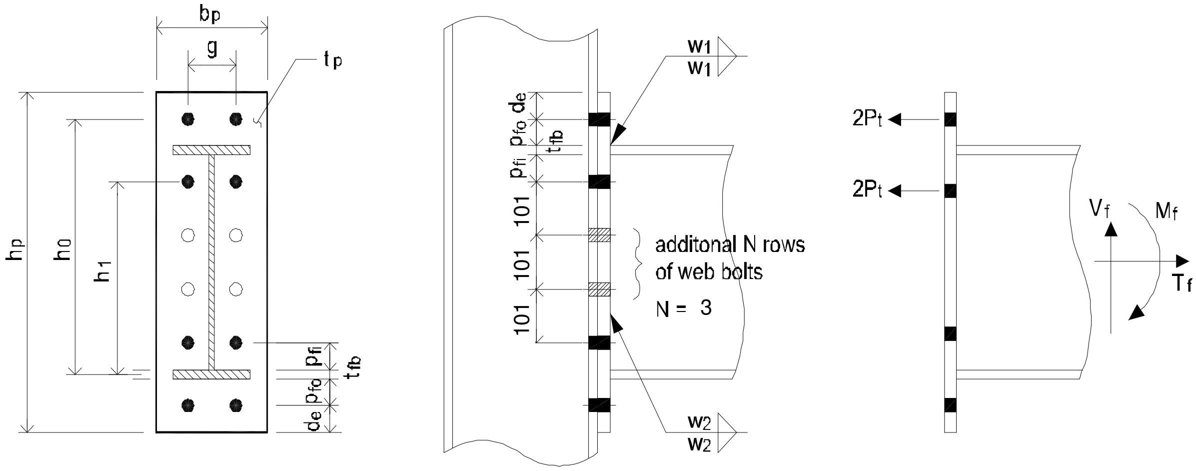
	Job No :		Chk :	Test
	Doc No :		Date :	2013-02-04
	Subject :	www.civilbay.com	Rev :	0

FOUR BOLT UNSTIFFENED MOMENT CONNECTION DESIGN

Four bolt unstiffened moment connection design based on	**Code Abbreviation**
CSA-S16-09 Limit States Design of Steel Structures	*CSA-S16-09*
AISC Design Guide 4: Extended End-Plate Moment Connections Seismic and Wind Applications	*AISC Design Guide 4*
AISC Design Guide 13: Wide-Flange Column Stiffening at Moment Connections	*AISC Design Guide 13*
AISC Steel Construction Manual 13th Edition	*AISC SCM 13th Ed*



INPUT — **Code Reference**

Bolt grade: A325 -N; Bolt diameter d_b = 1 [in] — *CSA-S16-09*

Bolt min tensile strength F_{ub} = 825 [MPa] — 13.12.1.2

Bolt hole is: Punched — Use Punched unless it's confirmed to be Drilled

Bolt thread is: Included — Use Included unless it's confirmed to be Excluded

Beam properties: W_610 W530x66

d_b =	525	[mm]	b_{fb} =	165	[mm]
t_{fb} =	11.4	[mm]	t_{wb} =	8.9	[mm]
k_b =	24.1	[mm]			

Column properties: W_310 W310x97 — b_{fc} >= b_p **OK**

d_c =	308	[mm]	b_{fc} =	305	[mm]
t_{fc} =	15.4	[mm]	t_{wc} =	9.9	[mm]
k_c =	30.5	[mm]	A =	12300	[mm^2]

W shape material strength	F_y =	345	[MPa]	F_u =	450	[MPa]
End plate material strength	F_{yp} =	300	[MPa]	F_{up} =	450	[MPa]

				suggest	*AISC Design Guide 4*
End plate width	b_p =	190	[mm]	only b_p = (b_{fb} + 1") used for design	Page 16
Gauge	g =	130	[mm]	130	
Bolt clear dist - inner bolt	p_{fi} =	50	[mm]	50	
Bolt clear dist - outer bolt	p_{fo} =	50	[mm]	50	
Bolt edge dist	d_e =	45	[mm]	45	
End plate thickness	t_p =	40.0	[mm]	25.4 or 31.8	

Check if MC located at top of column **Code Reference**

Dist between beam top flange & top of column <= d_c No MC is NOT located at top of column

Factored beam moment	M_{con} =	268	[kNm]						
Factored beam tensile force	T_f =	25	[kN]						
Moment converted from tension	M_{ten} =	$0.5T_f \times (d_b - t_{fb})$				= 6.4	[kNm]		
Factored moment for design	$\mathbf{M_f}$ =	$M_{con} + M_{ten}$				= **274**	[kNm]		
Factored beam shear	$\mathbf{V_f}$ =	259	[kN]			= **259**	[kN]		
Factored column axial compressioi	$\mathbf{C_f}$ =	0	[kN]			= **0**	[kN]		
Weld electrode	=	E49XX			X_u	= 490	[MPa]		
				suggest					
Fillet weld - beam flange	w_1 =	10	[mm]	10 mm					
Fillet weld - beam web	w_2 =	8	[mm]	8 mm					
Min. fillet weld size	=	8	[mm]		ratio	= 1.00		OK	*CSA-W59-03* Table 4.4
Min edge distance	=	44	[mm]		ratio	= 0.98		OK	*CSA-S16-09* Table 6
Resistance factor	ϕ =	0.90			ϕ_u	= 0.75			13.1 (a)
	ϕ_b =	0.80			ϕ_{br}	= 0.80			13.1 (c) (g)
	ϕ_w =	0.67							13.1 (h)

CONCLUSION

Overall	ratio =	**1.00**	OK
Bolt			
Bolt Tension	ratio =	0.53	OK
Bolt Shear	ratio =	0.31	OK
Bolt Bearing & Tear Out on End Plate	ratio =	0.08	OK
Bolt Bearing & Tear Out on Column Flange	ratio =	0.15	OK
Weld			
Beam Flange To End Plate Fillet Weld - Tension	ratio =	0.82	OK
Beam Web To End Plate Fillet Weld - Shear	ratio =	0.41	OK
End Plate			
End Plate Thickness by Yield Line Method	ratio =	0.46	OK
End Plate Thickness Considering Prying Action	ratio =	0.53	OK
End Plate Shear Yielding Subject To Flange Tension Force	ratio =	0.22	OK
End Plate Shear Rupture Subject To Flange Tension Force	ratio =	0.25	OK
End Plate Shear Yielding Subject To Vertical Shear	ratio =	0.03	OK
End Plate Shear Rupture Subject To Vertical Shear	ratio =	0.03	OK
End Plate Block Shear	ratio =	0.06	OK
Column			
Column Flange Flexural Yeilding	ratio =	1.00	OK
Column Web Yeilding - Tension & Compression	ratio =	0.53	OK
Column Web Buckling - Compression	ratio =	0.76	OK
Column Web Crippling - Compression	ratio =	0.76	OK
Column Panel Zone Web Shear	ratio =	0.94	OK

DESIGN CHECK — **Code Reference**

For net area calculation — *CSA-S16-09*

1. Bolt hole dia is 2mm larger than nominal bolt dia
2. For punched hole, addtional 2mm is added to bolt hole dia when it's used in net area width calc — 12.3.2

d_h = 29.4 [mm] for Punched hole

Tension bolt lever arm h_0 = 569.3 [mm] h_1 = 457.9 [mm]

Bolt

Bolt Tension

Nominal bolt area	A_b =		= 506.7	[mm^2]	
Single bolt tensile resistance	P_t =	$0.75\ \phi_b\ A_b\ F_{ub}$	= 250.8	[kN]	13.12.1.3
Moment resistance by bolt	M_r =	$2 \times P_t \times (h_0 + h_1)$	= 515	[kNm]	
	ratio =	0.53	> M_f	**OK**	

Bolt Shear — *AISC Design Guide 4*

Assume all shear taken by compression side bolts only — Page 9 2.1-4

Bolt number taking shear n_{bv} = 6 shear plane m = 1 — *CSA-S16-09*

Bolt threads are intercepted by a shear plane

A_b =	$0.7\ A$	= **354.7**	[mm^2]	13.12.1.2 (c)
V_r =	$0.6\ \phi_b\ n_{bv}\ m\ A_b\ F_{ub}$	= 843	[kN]	13.12.1.2 (c)
ratio =	0.31	> V_f	**OK**	

Bolt Bearing & Tear Out on End Plate

Bearing strength per bolt	n_{bv} = 1	bolt dia d	= 25.4	[mm]	
	B_r =	$3\ \phi_{br}\ n_{bv}\ t_p\ d\ F_{up}$	= 1097	[kN]	13.12.1.2 (a)
Exterior bolt tear out strength per bolt					
Gross shear area	A_{gv} =	$d_e \times t_p \times 2$ side	= 3600	[mm^2]	
Tear-out resistance per bolt	T_{r1} =	$\phi_u 0.6 A_{gv} \frac{F_y + F_u}{2}$	= 608	[kN]	13.11
Exterior bolt shear resistance	V_{r1} =	$\min(B_r, T_{r1})$	= 608	[kN]	
Interior bolt tear out strength per bolt					
Gross shear area	A_{gv} =	$(p_{fi} + t_{fb} + p_{fo}) \times t_p \times 2$ side	= 8912	[mm^2]	
Tear-out resistance per bolt	T_{r2} =	$\phi_u 0.6 A_{gv} \frac{F_y + F_u}{2}$	= 1504	[kN]	13.11
Interior bolt shear resistance	V_{r2} =	$\min(B_r, T_{r2})$	= 1097	[kN]	
Total shear resistance	V_r =	$V_{r1} \times 2 + V_{r2} \times 2$	= 3410	[kN]	
	ratio =	0.08	> V_f	**OK**	

Bolt Bearing & Tear Out on Column Flange					**Code Reference**
Bearing strength per bolt	n_{bv} = 1	bolt dia d	= 25.4	[mm]	*CSA-S16-09*
	B_r = 3 ϕ_{br} n_{bv} t_{fc} d F_u		= 422	[kN]	13.12.1.2 (a)

Assume exterior bolt edge distance on column flange is big and tear-out not governing

Exterior bolt shear resistance	V_{r1} = B_r	= 422	[kN]	
Interior bolt tear out strength per bolt				
Gross shear area	A_{gv} = (p_{fi} + t_{fb} + p_{fo}) x t_{fc} x 2 side	= 3431	[mm^2]	
Tear-out resistance per bolt	T_{r2} = $\phi_u 0.6 A_{gv} \frac{F_y + F_u}{2}$	= 614	[kN]	13.11
Interior bolt shear resistance	V_{r2} = min (B_r, T_{r2})	= 422	[kN]	
Total shear resistance	V_r = V_{r1} x 2 + V_{r2} x 2	= 1690	[kN]	
	ratio = 0.15	> V_f	**OK**	

Weld

Fillet weld resistance				
Base metal resistance	A_m = w_1 x 1mm	= 10.0	[mm^2]	
	v_{rm} = 0.67 ϕ_w A_m F_{up}	= 2.0	[kN/mm]	13.13.2.2
Weld metal resistance	A_w = 0.707 x w_1 x 1mm	= 7.1	[mm^2]	
Angle of weld axis and force	θ =	= 0		
	v_{rw} = 0.67 ϕ_w A_w X_u (1+ 0.5 sinθ ^1.5)	= 1.6	[kN/mm]	13.13.2.2

Beam Flange To End Plate Fillet Weld - Tension				*AISC Design Guide 4*
For wind and low-seismic applications, the flange force used for flange weld design shall not be less than 0.6F_y A_{fb}				Page 38
60% of beam flange yield strength	P_{uf1} = 0.6 x F_y x b_{fb} x t_{fb}	= 389	[kN]	
Flange force by moment	P_{uf2} = M_f / (d_b - t_{fb})	= 534	[kN]	
Flange force used for design	P_{uf} = max (P_{uf1} , P_{uf2})	= **534**	[kN]	
Angle of weld axis and force	θ =	= 90		
Fillet weld resistance	v_r = min(v_{rm} , 1.5v_{rw})	= 2.02	[kN/mm]	
	V_r = v_r x (2 x b_{fb} - t_{wb})	= 649	[kN]	
	ratio = 0.82	> P_{uf}	**OK**	

Beam Web To End Plate Fillet Weld - Shear				
Angle of weld axis and force	θ =	= 0		
Fillet weld resistance	v_r = min(v_{rm} , v_{rw}) x w_2 / w_1	= 1.2	[kN/mm]	
Shear weld length	L_1 = 0.5d_b - t_{fb}	= 251.1	[mm]	Page 33
	L_2 = d_b - 2t_{fb} - p_{fi} - 2d_b	= 401.4	[mm]	
	V_r = v_r x min(L_1 , L_2) x 2 side	= 625	[kN]	
	ratio = 0.41	> V_f	**OK**	

Code Reference

End Plate — *AISC Design Guide 4*

End Plate Thickness by Yield Line Method

s = 0.5 x sqrt(b_p x g) = 78.7 [mm] Table 3-1

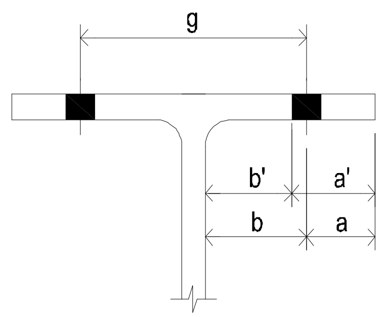
p_{fi} = s if p_{fi} > s = 50.0 [mm]

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}}+\frac{1}{s}\right)+\frac{h_0}{p_{fo}}-\frac{1}{2}\right]+\frac{2}{g}\left[h_1(p_{fi}+s)\right] = 3368.7 \text{ [mm]}$$

$$t_{p\ reqd} = \sqrt{\frac{1.11\ M_f}{\phi\ F_{yp} Y}} = 18.3 \text{ [mm]}$$ Page 21 (3.10)

ratio = 0.46 < t_p **OK**

End Plate Thickness Considering Prying Action — *AISC SCM 13th Ed*



	a = 1.19 [in]				
	b = 2.38 [in]				
Bolt dia	d_b = 1.000 [in]				
Bolt hole	d' = 1.063 [in]	F_u = 65	[ksi]		
Bolt ver. tributary length	p = (p_{fi} + 0.5t_{fb}) x 2	= 4.39	[in]		
	$a' = (a+0.5d_b) \le (1.25b+0.5d_b)$	= 1.69	[in]	page 9-12	
	b' = b - 0.5d_b	= 1.88	[in]	page 9-11	
	ρ = b' / a'	= 1.12			
	δ = 1 - d' / p	= 0.758		page 9-11	
End plate thickness	t = t_p	= 1.575	[in]		

From *AISC Design Guide 4* page 9 design assumption 4, all the shear force is assumed to be resisted by the compression side bolts. so there is no bolt tensile capacity reduction due to presence of shear

Tensile force per bolt without pryin	B = P_t	= 56.4	[kips]	
To get full B required thickness	$t_c = \sqrt{\frac{4.44\ B\ b'}{p\ F_u}}$	= 1.28	[in]	page 9-12
	$\alpha' = \frac{1}{\delta(1+\rho)}\left[(t_c/t)^2-1\right]$	= -0.21		page 9-13
Multiplier for prying action	Q =	= 1.000		page 9-12
Tensile force per bolt available	T_{avail} = B x Q	= 56.4	[kips]	

Code Reference

	$P_{t\text{-}pry} = T_{avail}$	= 251.0	[kN]		
Moment resistance by bolt tension	$M_r = 2 \times P_{t\text{-}pry} \times (h_0 + h_1)$	= 515.7	[kNm]		
	ratio = 0.53	> M_f	**OK**		

<u>End Plate Shear Yielding Subject To Flange Tension Force</u> — *AISC Design Guide 4*

Factored flange force for design	$P_{uf} = M_f / (d_b - t_{fb})$	= **534.3**	[kN]	Page 21 - item 8
Shear yielding resistance	$= \phi\, 0.6\, F_{yp}\, b_p\, t_p$	= 1233.8	[kN]	
	ratio = 0.22	> $0.5P_{uf}$	**OK**	

<u>End Plate Shear Rupture Subject To Flange Tension Force</u>

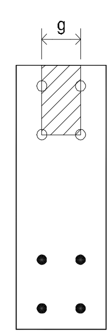
End plate net area	$A_n = (b_p - 2\, d_h) \times t_p$	= 5264	[mm^2]	Page 21 - item 9
Shear rupture resistance	$= \phi_u\, 0.6\, F_{up}\, A_n$	= 1066.0	[kN]	
	ratio = 0.25	> $0.5P_{uf}$	**OK**	

<u>End Plate Shear Yielding Subject To Vertical Shear</u>

End plate length	h_p = 715.0 [mm] — bolt hole d_h	= 29.4	[mm]
Shear yielding resistance	$= \phi\, 0.6\, F_{yp}\, h_p\, t_p \times 2$	= 9266.4	[kN]
	ratio = 0.03	> V_f	**OK**

<u>End Plate Shear Rupture Subject To Vertical Shear</u>

End plate net shear area	$A_n = (h_p - 4 \times d_h) \times t_p \times 2$	= 47792	[mm^2]
Shear rupture resistance	$= \phi_u\, 0.6\, F_{up}\, A_n$	= 9677.9	[kN]
	ratio = 0.03	> V_f	**OK**



<u>End Plate Block Shear</u> — *CSA-S16-09*

Net area in tension	$A_n = (g - d_h) \times t_p$	= 4024	[mm^2]	12.3.1 (a)
Gross area in shear	$A_{gv} = (p_{fi} + t_{fb} + p_{fo} + d_e) \times t_p \times 2$ side	= 19712	[mm^2]	
	$U_t = 1.0$			13.11 (a)
	$V_r = \phi_u \left[U_t A_n F_u + 0.6 A_{gv} \frac{F_y + F_u}{2} \right]$	= 4684	[kN]	13.11
	ratio = 0.06	> V_f	**OK**	

Column — **Code Reference**

Column Flange Flexural Yeilding — *AISC Design Guide 4*

s = 0.5 x sqrt(b_{fc} x g) = 99.6 [mm]

c = p_{fo} + t_{fb} + p_{fi} = 111.4 [mm]

$$Y_c = \frac{b_{fc}}{2}\left(\frac{h_1}{s}+\frac{h_0}{s}\right)+\frac{2}{g}\left[h_1\left(s+\frac{3c}{4}\right)+h_0\left(s+\frac{c}{4}\right)+\frac{c^2}{2}\right]+\frac{g}{2}$$

= 4139.7 [mm] — Table 3-4

$$t_{p\ reqd} = \sqrt{\frac{1.11\ M_f}{\phi\ F_{yc} Y_c}}$$ = 15.4 [mm] — Page 21 (3.10)

ratio = 1.00 < t_{fc} **OK**

column stiffeners not required

Column Web Yeilding - Tension & Compression

MC is NOT at top of column C_t = 1.0 ϕ = 1.0 — Page 22 - item 16

Bearing length N = t_{fb} + 2 x w_1 = 31.4 [mm]

Column web yeilding resistance $= \phi\ C_t(6k_c + N + 2t_p)\ F_{yc} t_{wc}$ = 1005 [kN]

ratio = 0.53 > P_{uf} **OK**

column stiffeners not required

Column Web Buckling - Compression

MC is NOT at top of column C_t = 1.0 ϕ = 0.9 — Page 22 - item 17

h = d_c - 2 x k_c = 247.0 [mm]

Column web buckling resistance $$= \phi\ C_t \frac{24 t_{wc}^3 \sqrt{E\ F_{yc}}}{h}$$ = 705 [kN]

ratio = 0.76 > P_{uf} **OK**

column stiffeners not required

Column Web Crippling - Compression

Bearing length N = t_{fb} + 2 x w_1 = 31.4 [mm] — Page 23 - item 18

ϕ = 0.75

Case 1: beam top flange located > 0.5d_c from end of column **This case applies**

Column web crippling resistance $$= \phi\ 0.8 t_{wc}^2 \left[1+3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$$ = 705 [kN]

Code Reference

Case 2: beam top flange located < $0.5d_c$ from end of column — *AISC Design Guide 4*

N / d_c = 0.102 — Page 23 - item 18

Case 2a For N/d_c <= 0.2 — **This case does NOT apply**

Column web crippling resistance $= \phi\, 0.4 t_{wc}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{EF_{yc}t_{fc}}{t_{wc}}}$ = 0 [kN]

Case 2b For N/d_c > 0.2 — **This case does NOT apply**

Column web crippling resistance $= \phi\, 0.4 t_{wc}^2 \left[1 + \left(\frac{4N}{d_c} - 0.2\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{EF_{yc}t_{fc}}{t_{wc}}}$ = 0 [kN]

Column web crippling resistance = = **705** [kN]

ratio = 0.76 > P_{uf} **OK**

column stiffeners not required

<u>Column Panel Zone Web Shear</u> — *AISC Design Guide 13*

Column min yield strength $P_y = F_y A$ = 4244 [kN]

C_f / P_y = 0.000 ϕ = 0.9

For N_f / P_y <= 0.4 **This case applies**

Panel zone shear resistance $R_v = \phi\, 0.6\, F_y\, d_c\, t_{wc}$ = 568 [kN] Page 6 (2.2-1)

For N_f / P_y > 0.4 **This case does NOT apply**

Panel zone shear resistance $R_v = \phi\, 0.6\, F_y\, d_c\, t_{wc}\, (1.4 - C_f / P_y)$ = 0 [kN] Page 6 (2.2-2)

Panel zone shear resistance R_v = = **568** [kN]

Factored flange force $P_{uf} = M_f / (d_b - t_{fb})$ = 534 [kN]

Neglect the effects of storey shear

Panel zone web shear force $V_u = P_{uf}$ = 534 [kN] Page 5 (2.1-5)

ratio = 0.94 < R_v **OK**

column web doubler plate not required

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	Job No :		Chk :	Test
	Doc No :		Date :	2013-02-04
	Subject :	www.civilbay.com	Rev :	0

FOUR BOLT UNSTIFFENED MOMENT CONNECTION DESIGN

Four bolt unstiffened moment connection design based on	**Code Abbreviation**
CSA-S16-09 Limit States Design of Steel Structures	*CSA-S16-09*
AISC Design Guide 4: Extended End-Plate Moment Connections Seismic and Wind Applications	*AISC Design Guide 4*
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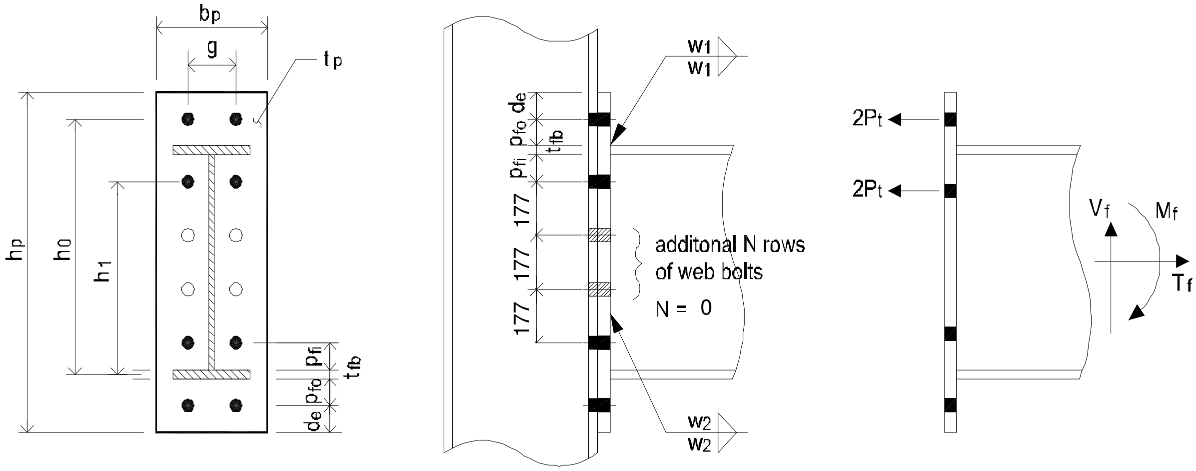

INPUT | **Code Reference**

Bolt grade	A325	-N	Bolt diameter d_b = 1 [in]		*CSA-S16-09*
Bolt min tensile strength	F_{ub} = 825	[MPa]			13.12.1.2
Bolt hole is	Punched	Use Punched unless it's confirmed to be Drilled			
Bolt thread is	Included	Use Included unless it's confirmed to be Excluded			

Beam properties	W_310		W310x60		
	d_b = 303	[mm]	b_{fb} = 203	[mm]	
	t_{fb} = 13.1	[mm]	t_{wb} = 7.5	[mm]	
	k_b = 25.9	[mm]			
Column properties	W_310		W310x143	b_{fc} >= b_p	**OK**
	d_c = 323	[mm]	b_{fc} = 309	[mm]	
	t_{fc} = 22.9	[mm]	t_{wc} = 14.0	[mm]	
	k_c = 38.1	[mm]	A = 18200	[mm^2]	
W shape material strength	F_y = 345	[MPa]	F_u = 450	[MPa]	
End plate material strength	F_{yp} = 300	[MPa]	F_{up} = 450	[MPa]	

			suggest	*AISC Design Guide 4*
End plate width	b_p = 228	[mm]	only b_p = (b_{fb} + 1") used for design	Page 16
Gauge	g = 130	[mm]	130	
Bolt clear dist - inner bolt	p_{fi} = 50	[mm]	50	
Bolt clear dist - outer bolt	p_{fo} = 50	[mm]	50	
Bolt edge dist	d_e = 45	[mm]	45	
End plate thickness	t_p = 40.0	[mm]	25.4 or 31.8	

Check if MC located at top of column **Code Reference**

Dist between beam top flange & top of column <= d_c No MC is NOT located at top of column

Factored beam moment	M_{con}	=	227	[kNm]				
Factored beam tensile force	T_f	=	25	[kN]				
Moment converted from tension	M_{ten}	=	$0.5T_f \times (d_b - t_{fb})$		=	3.6	[kNm]	
Factored moment for design	$\mathbf{M_f}$	=	$M_{con} + M_{ten}$		=	**231**	[kNm]	
Factored beam shear	$\mathbf{V_f}$	=	259	[kN]	=	**259**	[kN]	
Factored column axial compressio	$\mathbf{C_f}$	=	0	[kN]	=	**0**	[kN]	
Weld electrode		=	E49XX		X_u =	490	[MPa]	
				suggest				
Fillet weld - beam flange	w_1	=	10	[mm] 12 mm				
Fillet weld - beam web	w_2	=	8	[mm] 8 mm				
Min. fillet weld size		=	8	[mm]	ratio =	1.00	OK	*CSA-W59-03* Table 4.4
Min edge distance		=	44	[mm]	ratio =	0.98	OK	*CSA-S16-09* Table 6
Resistance factor	ϕ	=	0.90		ϕ_u =	0.75		13.1 (a)
	ϕ_b	=	0.80		ϕ_{br} =	0.80		13.1 (c) (g)
	ϕ_w	=	0.67					13.1 (h)

CONCLUSION

Overall	ratio =	**0.99**	OK
Bolt			
Bolt Tension	ratio =	0.79	OK
Bolt Shear	ratio =	0.46	OK
Bolt Bearing & Tear Out on End Plate	ratio =	0.08	OK
Bolt Bearing & Tear Out on Column Flange	ratio =	0.10	OK
Weld			
Beam Flange To End Plate Fillet Weld - Tension	ratio =	0.99	OK
Beam Web To End Plate Fillet Weld - Shear	ratio =	0.75	OK
End Plate			
End Plate Thickness by Yield Line Method	ratio =	0.54	OK
End Plate Thickness Considering Prying Action	ratio =	0.79	OK
End Plate Shear Yielding Subject To Flange Tension Force	ratio =	0.27	OK
End Plate Shear Rupture Subject To Flange Tension Force	ratio =	0.29	OK
End Plate Shear Yielding Subject To Vertical Shear	ratio =	0.04	OK
End Plate Shear Rupture Subject To Vertical Shear	ratio =	0.04	OK
End Plate Block Shear	ratio =	0.07	OK
Column			
Column Flange Flexural Yeilding	ratio =	0.81	OK
Column Web Yeilding - Tension & Compression	ratio =	0.48	OK
Column Web Buckling - Compression	ratio =	0.40	OK
Column Web Crippling - Compression	ratio =	0.56	OK
Column Panel Zone Web Shear	ratio =	0.94	OK

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	Doc No :			Date :	2013-02-04
Calculation Sheet	Subject :	www.civilbay.com		Rev :	0

DESIGN CHECK — **Code Reference**

For net area calculation — *CSA-S16-09*

1. Bolt hole dia is 2mm larger than nominal bolt dia
2. For punched hole, addtional 2mm is added to bolt hole dia when it's used in net area width calc — 12.3.2

d_h = 29.4 [mm] for Punched hole

Tension bolt lever arm h_0 = 346.4 [mm] h_1 = 233.3 [mm]

Bolt

Bolt Tension

Nominal bolt area	A_b =		= 506.7	[mm^2]	
Single bolt tensile resistance	P_t =	$0.75\ \phi_b\ A_b\ F_{ub}$	= 250.8	[kN]	13.12.1.3
Moment resistance by bolt	M_r =	$2 \times P_t \times (h_0 + h_1)$	= 291	[kNm]	
	ratio =	0.79	> M_f	**OK**	

Bolt Shear — *AISC Design Guide 4*

Assume all shear taken by compression side bolts only — Page 9 2.1-4

Bolt number taking shear n_{bv} = 4 shear plane m = 1 — *CSA-S16-09*

Bolt threads are intercepted by a shear plane

A_b =	0.7 A	= **354.7**	[mm^2]	13.12.1.2 (c)
V_r =	$0.6\ \phi_b\ n_{bv}\ m\ A_b\ F_{ub}$	= 562	[kN]	13.12.1.2 (c)
ratio =	0.46	> V_f	**OK**	

Bolt Bearing & Tear Out on End Plate

Bearing strength per bolt	n_{bv} = 1	bolt dia d	= 25.4	[mm]	
	B_r =	$3\ \phi_{br}\ n_{bv}\ t_p\ d\ F_{up}$	= 1097	[kN]	13.12.1.2 (a)
Exterior bolt tear out strength per bolt					
Gross shear area	A_{gv} =	$d_e \times t_p \times 2$ side	= 3600	[mm^2]	
Tear-out resistance per bolt	T_{r1} =	$\phi_u 0.6 A_{gv} \frac{F_y + F_u}{2}$	= 608	[kN]	13.11
Exterior bolt shear resistance	V_{r1} =	min (B_r, T_{r1})	= 608	[kN]	
Interior bolt tear out strength per bolt					
Gross shear area	A_{gv} =	$(p_{fi} + t_{fb} + p_{fo}) \times t_p \times 2$ side	= 9048	[mm^2]	
Tear-out resistance per bolt	T_{r2} =	$\phi_u 0.6 A_{gv} \frac{F_y + F_u}{2}$	= 1527	[kN]	13.11
Interior bolt shear resistance	V_{r2} =	min (B_r, T_{r2})	= 1097	[kN]	
Total shear resistance	V_r =	$V_{r1} \times 2 + V_{r2} \times 2$	= 3410	[kN]	
	ratio =	0.08	> V_f	**OK**	

Bolt Bearing & Tear Out on Column Flange					**Code Reference**
Bearing strength per bolt	n_{bv} = 1	bolt dia d = 25.4		[mm]	*CSA-S16-09*
	B_r = $3\ \phi_{br}\ n_{bv}\ t_{fc}\ d\ F_u$	= 628		[kN]	13.12.1.2 (a)

Assume exterior bolt edge distance on column flange is big and tear-out not governing

Exterior bolt shear resistance	V_{r1} = B_r	= 628	[kN]	

Interior bolt tear out strength per bolt

Gross shear area	A_{gv} = $(p_{fi} + t_{fb} + p_{fo})$ x t_{fc} x 2 side	= 5180	[mm^2]	
Tear-out resistance per bolt	$T_{r2} = \phi_u 0.6 A_{gv} \dfrac{F_y + F_u}{2}$	= 927	[kN]	13.11
Interior bolt shear resistance	V_{r2} = min (B_r, T_{r2})	= 628	[kN]	
Total shear resistance	V_r = V_{r1} x 2 + V_{r2} x 2	= 2513	[kN]	
	ratio = 0.10	> V_f	**OK**	

Weld

Fillet weld resistance

Base metal resistance	A_m = w_1 x 1mm	= 10.0	[mm^2]	
	v_{rm} = $0.67\ \phi_w\ A_m\ F_{up}$	= 2.0	[kN/mm]	13.13.2.2
Weld metal resistance	A_w = 0.707 x w_1 x 1mm	= 7.1	[mm^2]	
Angle of weld axis and force	θ =	= 0		
	v_{rw} = $0.67\ \phi_w\ A_w\ X_u$ (1+ 0.5 sinθ ^1.5)	= 1.6	[kN/mm]	13.13.2.2

Beam Flange To End Plate Fillet Weld - Tension — *AISC Design Guide 4*

For wind and low-seismic applications, the flange force used for flange weld design shall not be less than $0.6F_y A_{fb}$ — Page 38

60% of beam flange yield strength	P_{uf1} = 0.6 x F_y x b_{fb} x t_{fb}	= 550	[kN]	
Flange force by moment	P_{uf2} = M_f / (d_b - t_{fb})	= 796	[kN]	
Flange force used for design	P_{uf} = max (P_{uf1} , P_{uf2})	= **796**	[kN]	
Angle of weld axis and force	θ =	= 90		
Fillet weld resistance	v_r = min(v_{rm} , $1.5v_{rw}$)	= 2.02	[kN/mm]	
	V_r = v_r x (2 x b_{fb} - t_{wb})	= 805	[kN]	
	ratio = 0.99	> P_{uf}	**OK**	

Beam Web To End Plate Fillet Weld - Shear

Angle of weld axis and force	θ =	= 0		
Fillet weld resistance	v_r = min(v_{rm} , v_{rw}) x w_2 / w_1	= 1.2	[kN/mm]	
Shear weld length	L_1 = $0.5d_b$ - t_{fb}	= 138.4	[mm]	Page 33
	L_2 = d_b - $2t_{fb}$ - p_{fi} - $2d_b$	= 176.0	[mm]	
	V_r = v_r x min(L_1 , L_2) x 2 side	= 344	[kN]	
	ratio = 0.75	> V_f	**OK**	

Code Reference

End Plate

AISC Design Guide 4

End Plate Thickness by Yield Line Method

$s = 0.5 \times \text{sqrt}(b_p \times g)$ = 86.2 [mm] Table 3-1

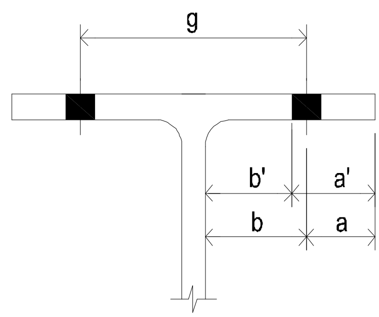
p_{fi} = s if $p_{fi} > s$ = 50.0 [mm]

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}} + \frac{1}{s}\right) + \frac{h_0}{p_{fo}} - \frac{1}{2}\right] + \frac{2}{g}\left[h_1(p_{fi} + s)\right] = 2065.3 \text{ [mm]}$$

$$t_{p\ reqd} = \sqrt{\frac{1.11\ M_f}{\phi\ F_{yp} Y}} = 21.4 \text{ [mm]}$$ Page 21 (3.10)

ratio = 0.54 < t_p **OK**

End Plate Thickness Considering Prying Action *AISC SCM 13th Ed*



a = 1.94 [in]

b = 2.41 [in]

Bolt dia d_b = 1.000 [in]

Bolt hole d' = 1.063 [in] F_u = 65 [ksi]

Bolt ver. tributary length p = (p_{fi} + $0.5t_{fb}$) x 2 = 4.45 [in]

$a' = (a + 0.5d_b) \le (1.25b + 0.5d_b)$ = 2.44 [in] page 9-12

$b' = b - 0.5d_b$ = 1.91 [in] page 9-11

$\rho = b' / a'$ = 0.78

$\delta = 1 - d' / p$ = 0.761 page 9-11

End plate thickness $t = t_p$ = 1.575 [in]

From *AISC Design Guide 4* page 9 design assumption 4, all the shear force is assumed to be resisted by the compression side bolts. so there is no bolt tensile capacity reduction due to presence of shear

Tensile force per bolt without pryin($B = P_t$ = 56.4 [kips]

To get full B required thickness $t_c = \sqrt{\frac{4.44\ B\ b'}{p\ F_u}}$ = 1.28 [in] page 9-12

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[(t_c / t)^2 - 1\right] = -0.25$$ page 9-13

Multiplier for prying action Q = = 1.000 page 9-12

Tensile force per bolt available $T_{avail} = B \times Q$ = 56.4 [kips]

Code Reference

$P_{t\text{-}pry} = T_{avail}$ = 251.0 [kN]

Moment resistance by bolt tension $M_r = 2 \times P_{t\text{-}pry} \times (h_0 + h_1)$ = 291.1 [kNm]

ratio = 0.79 > M_f **OK**

End Plate Shear Yielding Subject To Flange Tension Force — *AISC Design Guide 4*

Factored flange force for design $P_{uf} = M_f / (d_b - t_{fb})$ = **795.5** [kN] — Page 21 - item 8

Shear yielding resistance $= \phi\, 0.6\, F_{yp}\, b_p\, t_p$ = 1480.0 [kN]

ratio = 0.27 > $0.5P_{uf}$ **OK**

End Plate Shear Rupture Subject To Flange Tension Force

End plate net area $A_n = (b_p - 2\, d_h) \times t_p$ = 6784 [mm^2] — Page 21 - item 9

Shear rupture resistance $= \phi_u\, 0.6\, F_{up}\, A_n$ = 1373.8 [kN]

ratio = 0.29 > $0.5P_{uf}$ **OK**

End Plate Shear Yielding Subject To Vertical Shear

End plate length h_p = 493.0 [mm] bolt hole d_h = 29.4 [mm]

Shear yielding resistance $= \phi\, 0.6\, F_{yp}\, h_p\, t_p \times 2$ = 6389.3 [kN]

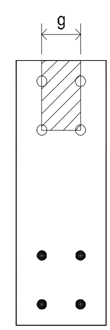
ratio = 0.04 > V_f **OK**

End Plate Shear Rupture Subject To Vertical Shear

End plate net shear area $A_n = (h_p - 4 \times d_h) \times t_p \times 2$ = 30032 [mm^2]

Shear rupture resistance $= \phi_u\, 0.6\, F_{up}\, A_n$ = 6081.5 [kN]

ratio = 0.04 > V_f **OK**



End Plate Block Shear — *CSA-S16-09*

Net area in tension $A_n = (g - d_h) \times t_p$ = 4024 [mm^2] — 12.3.1 (a)

Gross area in shear $A_{gv} = (p_{fi} + t_{fb} + p_{fo} + d_e) \times t_p \times 2$ side = 12648 [mm^2]

U_t = 1.0 — 13.11 (a)

$$V_r = \phi_u \left[U_t A_n F_u + 0.6 A_{gv} \frac{F_y + F_u}{2} \right]$$ = 3492 [kN] — 13.11

ratio = 0.07 > V_f **OK**

Column — **Code Reference**

Column Flange Flexural Yeilding — *AISC Design Guide 4*

$s = 0.5 \times \text{sqrt}(b_{fc} \times g)$ = 100.2 [mm]

$c = p_{fo} + t_{fb} + p_{fi}$ = 113.1 [mm]

$$Y_c = \frac{b_{fc}}{2}\left(\frac{h_1}{s}+\frac{h_0}{s}\right)+\frac{2}{g}\left[h_1\left(s+\frac{3c}{4}\right)+h_0\left(s+\frac{c}{4}\right)+\frac{c^2}{2}\right]+\frac{g}{2}$$

= 2406.4 [mm] — Table 3-4

$$t_{p\ reqd} = \sqrt{\frac{1.11\ M_f}{\phi\ F_{yc} Y_c}}$$ = 18.5 [mm] — Page 21 (3.10)

ratio = 0.81 $< t_{fc}$ **OK**

column stiffeners not required

Column Web Yeilding - Tension & Compression

MC is NOT at top of column C_t = 1.0 ϕ = 1.0 — Page 22 - item 16

Bearing length $N = t_{fb} + 2 \times w_1$ = 33.1 [mm]

Column web yeilding resistance $= \phi\, C_t (6k_c + N + 2t_p)\, F_{yc} t_{wc}$ = 1650 [kN]

ratio = 0.48 $> P_{uf}$ **OK**

column stiffeners not required

Column Web Buckling - Compression

MC is NOT at top of column C_t = 1.0 ϕ = 0.9 — Page 22 - item 17

$h = d_c - 2 \times k_c$ = 246.8 [mm]

Column web buckling resistance $$= \phi\, C_t \frac{24 t_{wc}^3 \sqrt{E\ F_{yc}}}{h}$$ = 1995 [kN]

ratio = 0.40 $> P_{uf}$ **OK**

column stiffeners not required

Column Web Crippling - Compression

Bearing length $N = t_{fb} + 2 \times w_1$ = 33.1 [mm] — Page 23 - item 18

ϕ = 0.75

Case 1: beam top flange located > $0.5d_c$ from end of column **This case applies**

Column web crippling resistance $$= \phi\ 0.8 t_{wc}^2 \left[1+3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$$ = 1433 [kN]

Code Reference

Case 2: beam top flange located < $0.5d_c$ from end of column — *AISC Design Guide 4*

$N / d_c = 0.102$ — Page 23 - item 18

Case 2a For $N/d_c <= 0.2$ — **This case does NOT apply**

Column web crippling resistance $= \phi\ 0.4 t_{wc}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{EF_{yc} t_{fc}}{t_{wc}}}$ = 0 [kN]

Case 2b For $N/d_c > 0.2$ — **This case does NOT apply**

Column web crippling resistance $= \phi\ 0.4 t_{wc}^2 \left[1 + \left(\frac{4N}{d_c} - 0.2\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{EF_{yc} t_{fc}}{t_{wc}}}$ = 0 [kN]

Column web crippling resistance = = **1433** [kN]

ratio = 0.56 > P_{uf} **OK**

column stiffeners not required

<u>Column Panel Zone Web Shear</u> — *AISC Design Guide 13*

Column min yield strength $P_y = F_y A$ = 6279 [kN]

$C_f / P_y = 0.000$ ϕ = 0.9

For $N_f / P_y <= 0.4$ **This case applies**

Panel zone shear resistance $R_v = \phi\ 0.6\ F_y\ d_c\ t_{wc}$ = 842 [kN] Page 6 (2.2-1)

For $N_f / P_y > 0.4$ **This case does NOT apply**

Panel zone shear resistance $R_v = \phi\ 0.6\ F_y\ d_c\ t_{wc}\ (1.4 - C_f / P_y)$ = 0 [kN] Page 6 (2.2-2)

Panel zone shear resistance R_v = = **842** [kN]

Factored flange force $P_{uf} = M_f / (d_b - t_{fb})$ = 796 [kN]

Neglect the effects of storey shear

Panel zone web shear force $V_u = P_{uf}$ = 796 [kN] Page 5 (2.1-5)

ratio = 0.94 < R_v **OK**

column web doubler plate not required