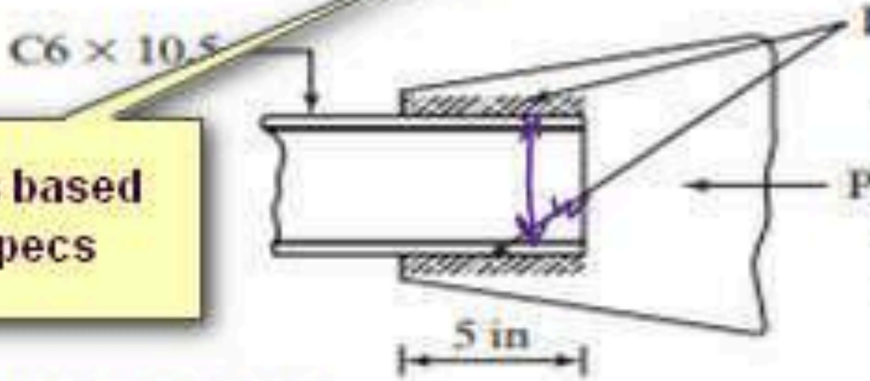
3-33. A C6 × 10.5 consisting of A36 steel with two longitudinal welds shown in Fig. P3-33. (*Ans.* LRFD 99.5 k, ASD 66.2 k)

Answer is based on 2010 specs



For channel

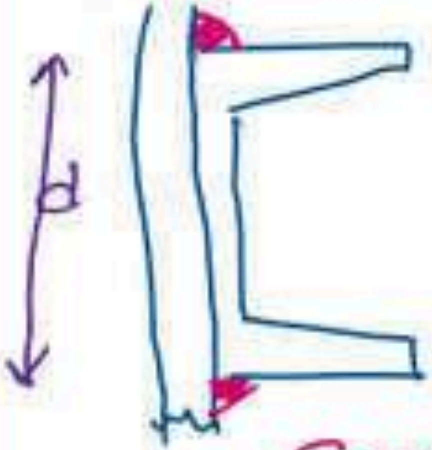
Determine the LRFD design strength and the ASD allowable strength for sections given –neglect block shear.

FIGURE P3-33

Solution → A36 : $F_y = 36$ ksi, $F_{ult} = 58$ ksi

w → Equals d height of C channel

$l_{weld} = l = 5''$



C6×10.50

Prepared by Eng.Maged Kamel.

→ CM # 14

Design is based on -2010 specs.

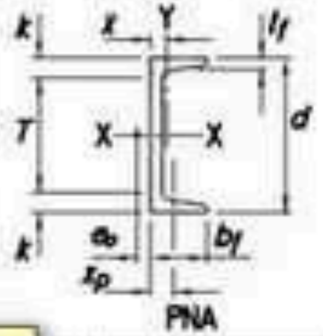


Table 1-5

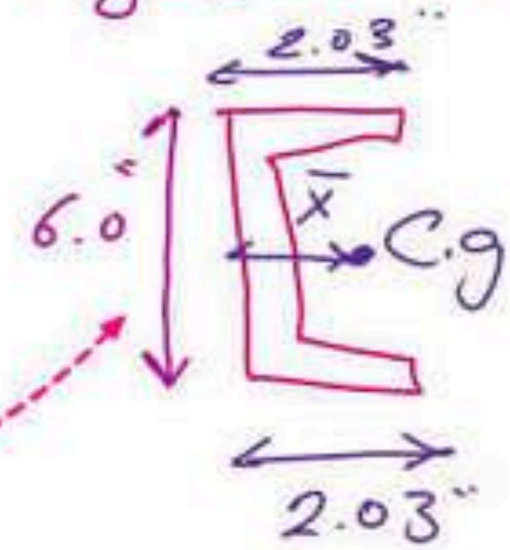
C-Shapes

Dimensions

C6x10.50

pe	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Average Thickness, t_f		Distance k	Distance T	Distance Workable Gage	r_{ts}	h_o
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
C6x13	3.82	6.00	6	0.437	7/16	1/4	2.16	2 1/8	0.343	5/16	13/16	4 3/8	1 3/8 g	0.689	5.66
x10.5	3.07	6.00	6	0.314	5/16	3/16	2.03	2	0.343	5/16	13/16	4 3/8	1 1/8 g	0.669	5.66
x8.2	2.39	6.00	6	0.200	3/16	1/8	1.92	1 7/8	0.343	5/16	13/16	4 3/8	1 1/8 g	0.643	5.66

$A_g = 3.07$ inch²



Data From Table 1-5

C6 x 10.50

$t_F = 0.343"$

→ CM # 14

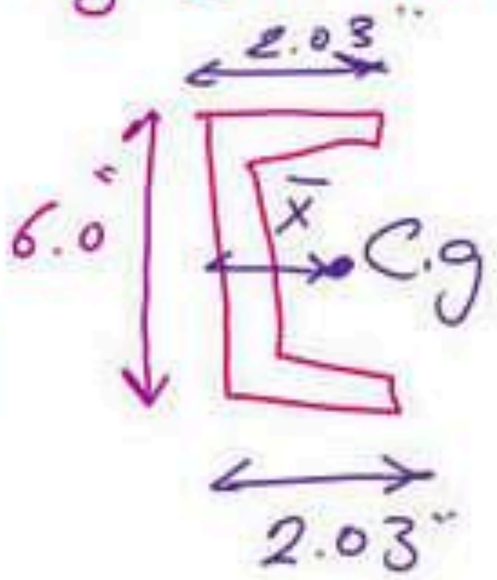
design is based on -2010 specs

Table 1-5-part 2

Table 1-5 (continued)
C-Shapes
Properties

Nominal Wt.	Shear Ctr, e_o	Axis X-X				Axis Y-Y						Torsional Properties			
		I	S	r	Z	I	S	r	$\bar{x}$	Z	x_p	J	C_w	$\bar{r}_o$	H
lb/ft	in.	in.[4]	in.[3]	in.	in.[3]	in.[4]	in.[3]	in.	in.	in.[3]	in.	in.[4]	in.[6]	in.	
13	0.380	17.3	5.78	2.13	7.29	1.05	0.638	0.524	0.514	1.35	0.318	0.237	7.19	2.37	0.858
→ 10.5	0.486	15.1	5.04	2.22	6.18	0.860	0.561	0.529	0.500	1.14	0.256	0.128	5.91	2.48	0.842
8.2	0.599	13.1	4.35	2.34	5.16	0.687	0.488	0.536	0.512	0.987	0.199	0.0736	4.70	2.65	0.824

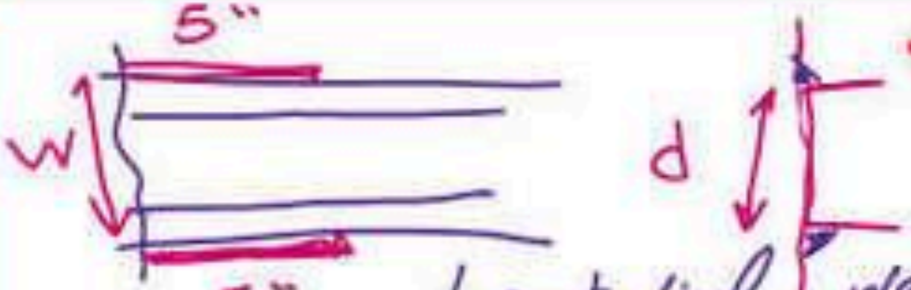
$A_g = 3.07$ inch^2



Data From Table 1-5 part-2

C6 x 10.50 $\bar{x} = 0.500$"

CM#14 - 2010 Specs



Longitudinal weld

This is Case-2: All members Except plate (not Case 4)

weld Length = 5" for one side

$w = 6" = d_C$ & $\bar{x} = 0.50"$

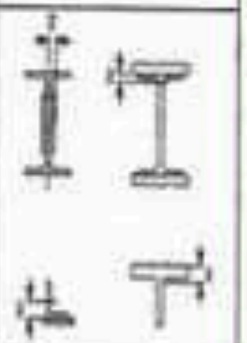
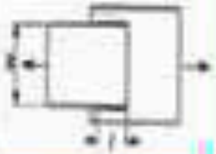
U value

from Case - 2

$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.50}{5} = 0.90$ ⇒ selected

case 2 is only applicable

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element	Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).	$U = 1.0$	—
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds or by longitudinal welds in combination with transverse welds. (Alternatively, for W, M, S and HP, Case 7 may be used. For angles, Case 8 may be used.)	$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.	$U = 1.0$ and A_n = area of the directly connected elements	—
4	Plates where the tension load is transmitted by longitudinal welds only. Plates only	$l \geq 2w$...$U = 1.0$ $2w > l \geq 1.5w$...$U = 0.87$ $1.5w > l \geq w$...$U = 0.75$	

Specs # 2010

A36

$F_y = 36$ ksi

$F_u = 58$ ksi

$U = 0.90$

$A_g = 3.07 \text{ inch}^2 = A_{net}$

$A_e = U A_{nt} = 0.90(3.07) = 2.763 \text{ inch}^2$

Tensile strength — Tensile yielding

$\phi = 0.90$

$\Omega = 1.67$

LRFD Design

$\phi P_n = 0.90(A_g \cdot F_y) = 0.90(3.07)(36) = 99.47$ kips

Tensile strength (rupture) $\Rightarrow \phi = 0.75$ & $\Omega = 2.0$

$P_n = F_{ult} \cdot A_e = 58 \cdot (2.763) = 160.254$ kips

$\phi P_n = 0.75(160.254) = 120.19$ kips

$\phi P_n = \min(99.47,\ 120.19) = 99.47 \approx 99.50$ kips

yielding controls

$U = 0.90$

Specs # 2010

A_{36}

$A_g = 3.07\ inch^2 = A_{net}$

$F_y = 36\ ksi$

$A_e = U A_{nt} = 0.90(3.07) = 2.763\ inch^2$

$F_u = 58\ ksi$

ASD design

Tensile strength

Tensile yielding

$\phi = 0.90$

$\Omega = 1.67$

ASD Design

$\frac{1}{\Omega} P_n = \frac{1}{\Omega}(A_g \cdot F_y) = \frac{1}{1.67}(3.07)(36) = 66.18$ kips

Tensile strength (rupture) $\Rightarrow \phi = 0.75$ & $\Omega = 2.0$

$\frac{1}{\Omega} P_n = F_{ult} \cdot A_e = 58\,(2.763) = 160.254$ kips

yielding Controls

$\frac{1}{\Omega} P_n = \frac{1}{2}(160.254) = 80.127$ kips

$P_n = \min(66.18, 80.127) = 66.18$ kips ≈ 66.20 kips

Prepared by Eng.Maged Kamel.

Case - 2

Longtudinal weld in combination with Transverse weld

CM#15 - 2016 Specs

U value based on CM#15

Case - 4

Case of Longtudinal weld

weld Length = 5" for one side

$w = d = 6"$ Channel depth

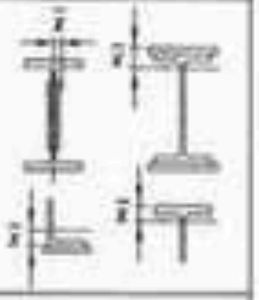
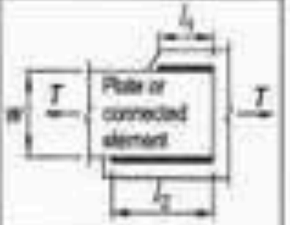
$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{L}\right) \Rightarrow \bar{x} = 0.5"$

Exprenion $\left(1 - \frac{\bar{x}}{L}\right) = 1$

$U = \frac{3(5)^2}{3(5)^2 + (6)^2} \cdot \left(1 - \frac{0.50}{5}\right) = \frac{75}{111}(0.90)$

$U = 0.608 \rightarrow$

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element	Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).	$U = 1.0$	–
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)	$U = 1 - \frac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.	$U = 1.0$ and A_n = area of the directly connected elements	–
4[a]	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.	$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$	

case 4 a for channels

by Eng.Maged Kamel.

CM #16

Case 4a -

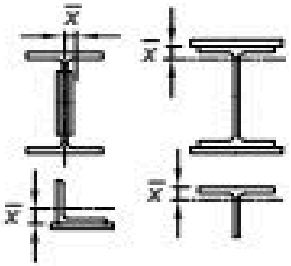
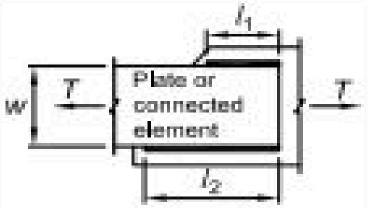
Matches with CM #15

Same Equation

$U = \frac{3L^2}{3L^2+W^2}\left(1-\frac{\bar{X}}{L}\right)$ *

Channels with Long. weld

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element	Shear Lag Factor, U	Examples
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5, and 6).	$U = 1.0$	–
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S, and HP shapes and Case 8 is permitted for angles.	$U = 1 - \frac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.	$U = 1.0$ and A_n = area of the directly connected elements	–
4[a]	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.	$U = \frac{3l^2}{3l^2+w^2}\left(1-\frac{\bar{x}}{l}\right)$	

get $1-\frac{\bar{X}}{L}$

$U = 0.608$

Specs # 2016

A_{36}

$F_y = 36$ ksi

$F_u = 58$ ksi

$A_g = 3.07$ inch^2 = A_{nt}

$A_e = U \cdot A_n = 0.608(3.07) = 1.867$ inch^2

Tensile strength

$\phi = 0.90$ yielding

$\Omega = 1.67$

LRFD Design

$\phi P_n = 0.90(36)(3.07) = 99.468$ Tensile yielding

Tensile strength (rupture) ⇒ $\phi = 0.75$ & $\Omega = 2.0$

$P_n = F_{ult} \cdot A_e = 58(1.867) = 108.286$ kips

$\phi P_n = 0.75(108.286) = 81.214$ kips

$\phi P_n = \min(99.468, 81.29) = 81.24$ kips → Rupture Controls

Prepared by Eng.Maged Kamel.

$U = 0.608$ Specs # 2016

$A_g = 3.07 \text{ inch}^2 = A_{nt}$

$A_e = U \cdot A_n = 0.608(3.07) = 1.867 \text{ inch}^2$

A36: $F_y = 36$ ksi, $F_u = 58$ ksi

Tensile strength

$\phi = 0.90$ yielding

$\Omega = 1.67$

ASD Design

$\frac{1}{\Omega} P_n = \frac{1}{1.67}(36)(3.07) = 66.18$ Tensile yielding

Tensile strength (rupture) $\Rightarrow \phi = 0.75$ & $\Omega = 2.0$

$P_n = F_{ult} \cdot A_e = 58(1.867) = 108.286$ kips

$\frac{1}{\Omega} P_n = \frac{1}{2}(108.286) = 54.143$ kips

$\min \frac{1}{\Omega} P_n = \min(66.18, 54.143) = 54.14$ kips → Rupture Controls

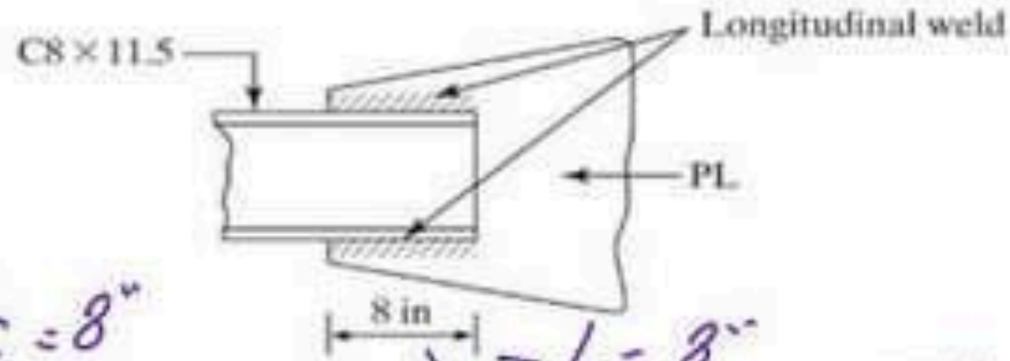
Prepared by Eng.Maged Kamel.

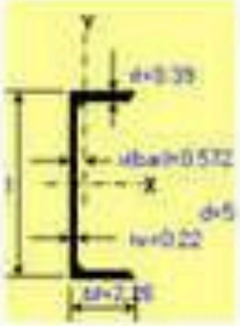
Mccormac

10. 3-33. A C8 × 11.5 consisting of A36 steel with two longitudinal welds shown in Fig. P3-33.

(*Ans.* LRFD 102.2 k; ASD 68.1 k)

Determine the LRFD design strength and the ASD allowable strength for sections given – neglect block shear- Sixth version-2016.





	C8X11.5	
A =	3.37	in^2
d =	8	in.
tw =	0.22	in.
bf =	2.26	in.
tf =	0.39	in.
[illegible] =	0.9375	in.
[illegible] =	0.9375	in.
T =	6.125	in.
ge =	1.375	in.
rts =	0.756	in.
ho =	7.61	in.
Wt. =	11.5	plf
eo =	0.697	in.
Ix =	32.5	in^4
Sx =	8.14	in^3
rx =	3.11	in.
Zx =	9.63	in^3
Iy =	1.31	in^4
Sy =	0.775	in^3
ry =	0.623	in.
x(bar) =	0.572	in.

$w = d_C = 8''$ → $L = 8''$

This is problem 3-33 in the 6th-edition

$A_g = 3.37\ inch^2$

$A_n = 3.37\ inch^2$

No bolts

$$U = \frac{3w^2}{(3w^2+L^2)}\left(1-\frac{\bar{x}}{L}\right) \Rightarrow CM\#15$$

$$U = \frac{3(8)^2}{3(8)^2+(8)^2}\left(1-\frac{0.572}{8}\right) = 0.75(0.9277) = 0.696$$

$A_e = U A_n$

$= 0.696(3.37) = 2.3456\ inch^2$

Prepared by Eng.Maged Kamel.

Yielding $A_g = 3.37 \text{ inch}^2$

$F_y = 36 \text{ ksi}$ $\phi_b = 0.90$

$\Omega_b = 1.67$

$\phi P_n \text{ yielding} = \phi_b A_g F_y = 0.9(3.37)(36) = 109.19 \text{ kips}$

Rupture $A_e = 2.35 \text{ inch}^2$ $\phi_b = 0.75$

$F_u = 58 \text{ ksi}$ $\Omega_b = 2.0$

$\phi_t P_n \text{ rupture} = 0.75(2.35)(58) = 102.23 \text{ kips}$ < 109.19 kips

⇒ Design governed by rupture

ASD yielding $\frac{1}{\Omega_t} A_g F_y = \frac{1}{1.67}(3.37)(36) = 72.65 \text{ kips}$

Rupture $\frac{1}{\Omega_t} A_e F_u = \frac{1}{2}(2.35)(58) = 68.15 \text{ kips}$ ⇒ Author solution

$< 72.65 \text{ kips}$

Prepared by Eng.Maged Kamel.