3-34. A $\frac{3}{8} \times 5$ plate consisting of A36 steel with two longitudinal welds as shown in Fig. P3-34.

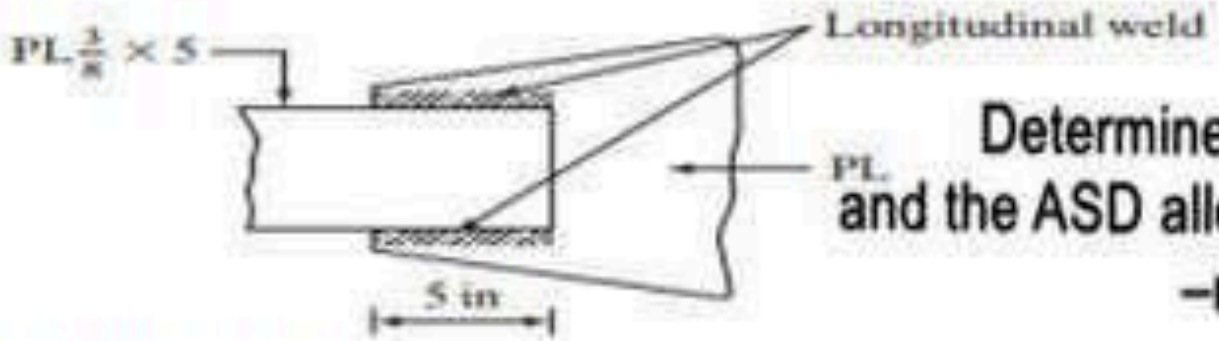
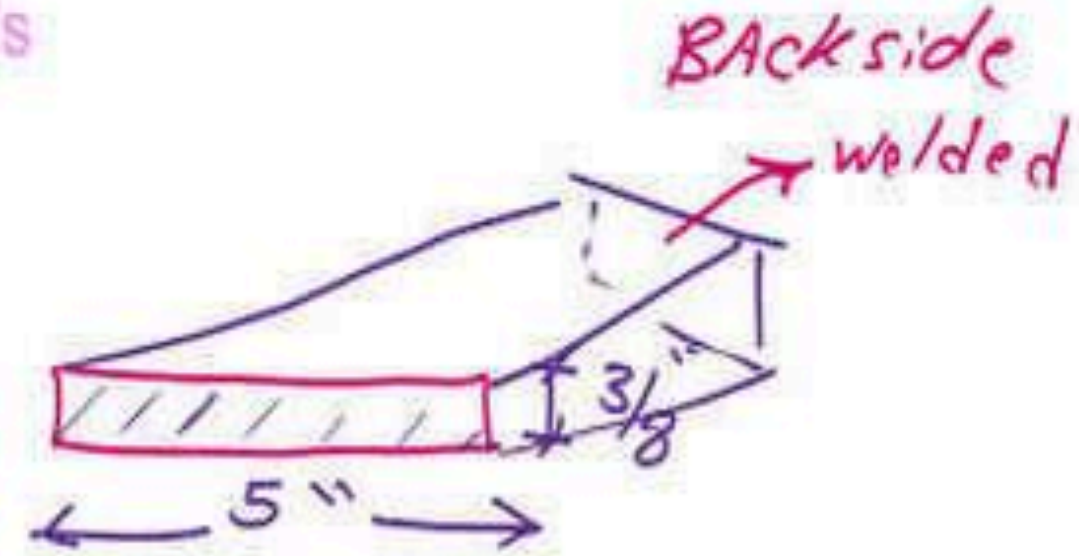


FIGURE P3-34

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

Case of Longtudinal welds



Solution:

A_{36} steel $\Rightarrow$ F_y : 36 ksi

F_u : 58 ksi

$A_{welded} = 2\left(\frac{3}{8}\right)(5) = 3.75 \text{ inch}^2$

CM# 14 - 2010 Specs

PL 3/8" x 5"

Case - 4

Case of Longtudinal weld

weld Length = 5" for one side

w = 5"

We have three cases

if $L \geq 2w$ $\quad \ell = 5"$ $\quad \frac{\ell}{w} = 1$

$U = 1.00$ $\quad 2w = 10"$

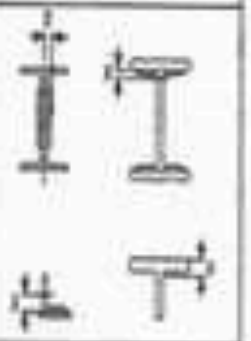
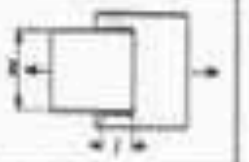
$2w < L \geq 1.50w \rightarrow U = 0.87$

$1.5w > L \geq w$ $\quad U = 0.75$

$\Rightarrow$ third Condition is applied

Case 4-2010

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.	$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	

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McCormac

As a result of the preceding information, the AISC Specification (D2) states that the nominal strength of a tension member, P_n, is to be the smaller of the values obtained by substituting into the following two expressions:

For the limit state of yielding in the gross section (which is intended to prevent excessive elongation of the member),

Yielding

$$P_n = F_y A_g \quad \text{(AISC Equation D2-1)}$$

Limit state of

$\phi_t P_n = \phi_t F_y A_g$ = design tensile strength by LRFD ($\phi_t = 0.9$)

$$\frac{P_n}{\Omega_t} = \frac{F_y A_g}{\Omega_t} = \text{allowable tensile strength for ASD } (\Omega_t = 1.67)$$

For tensile rupture in the net section, as where bolt or rivet holes are present,

Rupture

$$P_n = F_u A_e \quad \text{(AISC Equation D2-2)}$$

Limit state of

$\phi_t P_n = \phi_t F_u A_e$ = design tensile rupture strength for LRFD ($\phi_t = 0.75$)

$$\frac{P_n}{\Omega_t} = \frac{F_u A_e}{\Omega_t} \text{ allowable tensile rupture strength for ASD } (\Omega_t = 2.00)$$

Specs # 2010

$U = 0.75$

A_{36}

$A_g = 5\left(\frac{3}{8}\right) = \frac{15}{8} = 1.875 \text{ inch}^2 = A_n$

$A_e = U \cdot A_n = 0.75(1.875) = 1.406 \text{ inch}^2$

$F_y = 36$ ksi

$F_u = 58$ ksi

Tensile strength

Tensile yielding

$\phi = 0.90$ yielding

LRFD Design

$\Omega = 1.67$

$\phi P_n = 0.90(36)(1.875) = 60.75$ kips Tensile yielding

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

$P_n = F_{ult} \cdot A_e = 58 \cdot (1.406) = 81.55$ kips

$\phi P_n = 0.75(81.55) = 61.16$ kips

$\phi P_n = \min(60.75, 61.16) = 60.75$ kips $\Rightarrow$ 61 kips

yielding Controls

Prepared by Rag Magad Kamel

$U = 0.75$ Specs # 2010

A_{36}

$F_y = 36$ ksi

$F_u = 58$ ksi

$A_g = 5\left(\frac{3}{8}\right) = \frac{15}{8} = 1.875\ inch^2 = A_n$

$A_e = U \cdot A_n = 0.75(1.875) = 1.406\ inch^2$

Tensile strength

$\phi = 0.90$ yielding

$\Omega = 1.67$

ASD design strength-Yielding

$\frac{1}{\Omega} P_n = \frac{1}{\Omega} A_g F_y = \frac{1}{1.67}(67.50) = 40.42$ kips

Tensile strength (rupture) $\Rightarrow$ $\Omega = 2.0$

ASD design strength-rupture

$P_n = F_{ult} \cdot A_e = 58 \cdot (1.406) = 81.55$ kips

$\frac{1}{\Omega} P_n = \frac{1}{2}(81.55) = 40.775$ kips $= 40.78$ kips

$P_n = \min(40.42, 40.78) = 40.42$ kips

Yielding Controls

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CM#15 - 2016 Specs

PL 3/8 x

Case-4

Case of Longtudinal weld

Weld Length = 5" For one side

w = 5"

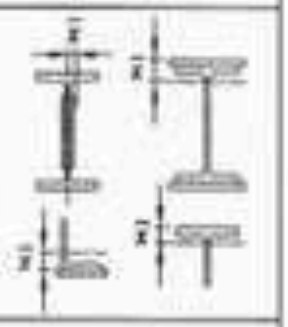
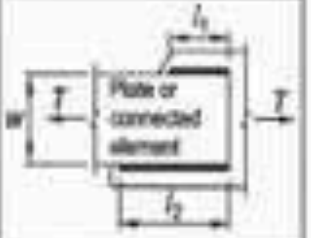
$$U = \frac{3l^2}{3l^2+w^2}\left(1-\frac{\bar{x}}{L}\right) \Rightarrow \bar{x}=0$$

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

$$U=\frac{3(5)^2}{3(5)^2+(5)^2}(1)=\frac{75}{100}=0.75$$

U value based on CM#15

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

	Description of Element	Shear Lag Factor, U	Example
1	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[4]	Plates, angles, channels with welds at heels, toes, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.	$U = \frac{3l^2}{3l^2+w^2}\left(1-\frac{\bar{x}}{l}\right)$	

Item 4a

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CM #16

Case 4a - Plates

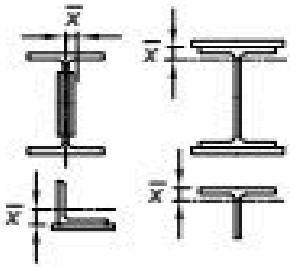
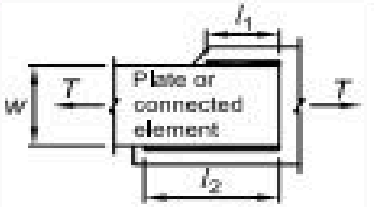
Matches

with CM #15

Same Equation

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

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

Specs # 2016 (1)

$U = 0.75$

$A_g = 5\left(\frac{3}{8}\right) = \frac{15}{8} = 1.875\ inch^2 = A_n$

$A_e = U \cdot A_n = 0.75(1.875) = 1.406\ inch^2$

A_{36}

$F_y = 36\ ksi$

$F_u = 58\ ksi$

Tensile strength

$\phi = 0.90$ yielding

$\Omega = 1.67$

LRFD Design

$\phi P_n = 0.90(36)(1.875) = 60.75$ kips Tensile yielding

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

$P_n = F_{ult} \cdot A_e = 58\,(1.406) = 81.55$ kips

$\phi P_n = 0.75(81.55) = 61.16$ kips

$\phi P_n = \min(60.75, 61.16) = 60.75$ kips $\Rightarrow$ 61 kips

yielding Controls

Prepared by Eng.Maged Kamel.

$U = 0.75$ Specs #2016 A_{36}

$A_g = 5\left(\frac{3}{8}\right) = \frac{15}{8} = 1.875\ inch^2 = A_n$

$A_e = U \cdot A_n = 0.75(1.875) = 1.406\ inch^2$

$F_y = 36\ ksi$

$F_u = 58\ ksi$

Tensile strength

$\phi = 0.90$ yielding $\Rightarrow A_g F_y = 1.875(36) = 67.50\ kips$

$\Omega = 1.67$

$\frac{1}{\Omega} P_n = \frac{1}{\Omega} A_g F_y = \frac{1}{1.67}(67.50) =$ **40.42 kips**

Tensile strength (rupture) $\Rightarrow$ $\Omega = 2.0$

$P_n = F_{ult} \cdot A_e = 58 \cdot (1.406) = 81.55\ kips$

$\frac{1}{\Omega} P_n = \frac{1}{2}(81.55) = 40.775\ kips =$ **40.78 kips**

$P_n = \min(40.42, 40.78) = 40.42\ kips$

yielding controls

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