CE 415
DESIGN OF STEEL STRUCTURES

LECTURE 4
TENSION MEMBER
LRFD

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OUTLINE

- Tension member design (LRFD)
- Property tables for angle and shear lag
- Design tension member
- Find capacity of tension member

LRFD Design

$\phi_t T_n \geq T_u$ Where,

ϕ_t = resistance factor relating to tension member strength
T_n = nominal strength of a tension member (see AISC-Chapter D)
$\phi_t T_n$ = design strength of a tension member
T_u = factored load on a tension member

Tension Members have THREE limit states:

1. Yielding on gross section
2. Fracture on effective section
3. *Block Shear*

Yielding on gross section

$$\phi_t T_n = \phi_t F_y A_g = 0.9 F_y A_g$$

Fracture on effective net section

$$\phi_t T_n = \phi_t F_u A_e = 0.75 F_u A_e$$

Note that the resistance factor ϕ_t is 0.90 for the yielding limit state and 0.75 for the fracture limit state.

Designation						A	I_x	r_x	y	I_y	r_y	x
L 9x4x1	9	x	4	x	1	12.0	97.0	2.84	3.50	12.0	1.00	1.00
L 7x4x0.625	7	x	4	x	5/8	6.48	32.4	2.24	2.46	7.84	1.10	0.963
L 6x4x0.75	6	x	4	x	3/4	6.94	24.5	1.88	2.08	8.68	1.12	1.08
L 6x4x0.625	6	x	4	x	5/8	5.86	21.1	1.90	2.03	7.52	1.13	1.03
L 6x4x0.5625	6	x	4	x	9/16	5.31	19.3	1.90	2.01	6.91	1.14	1.01
L 6x4x0.5	6	x	4	x	1/2	4.75	17.4	1.91	1.99	6.27	1.15	0.987
L 6x4x0.4375	6	x	4	x	7/16	4.18	15.5	1.92	1.96	5.60	1.16	0.964
L 6x4x0.375	6	x	4	x	3/8	3.61	13.5	1.93	1.94	4.90	1.17	0.941
L 4x3.5x0.437	4	x	3 1/2	x	7/16	3.09	4.76	1.24	1.23	3.40	1.05	0.978
L 4x3.5x0.375	4	x	3 1/2	x	3/8	2.67	4.18	1.25	1.21	2.95	1.06	0.955
L 4x3.5x0.312	4	x	3 1/2	x	5/16	2.25	3.56	1.26	1.18	2.55	1.07	0.932
L 4x3.5x0.25	4	x	3 1/2	x	1/4	1.81	2.91	1.27	1.16	2.09	1.07	0.909
L 4x3x0.625	4	x	3	x	5/8	3.98	6.03	1.23	1.37	2.87	0.849	0.871
L 4x3x0.5	4	x	3	x	1/2	3.25	5.05	1.25	1.33	2.42	0.864	0.827
L 4x3x0.4375	4	x	3	x	7/16	2.87	4.52	1.25	1.30	2.18	0.871	0.804
L 4x3x0.375	4	x	3	x	3/8	2.48	3.96	1.26	1.28	1.92	0.879	0.782
L 3.5x2.5x0.2	3 1/2	x	2 1/2	x	1/4	1.44	1.8	1.12	1.11	0.777	0.735	0.614
L 3x2.5x0.5	3	x	2 1/2	x	1/2	2.50	2.08	0.913	1.00	1.30	0.722	0.750
L 3x2.5x0.437	3	x	2 1/2	x	7/16	2.21	1.88	0.920	0.978	1.18	0.729	0.728
L 3x2.5x0.375	3	x	2 1/2	x	3/8	1.92	1.66	0.928	0.956	1.04	0.736	0.706
L 3x2.5x0.312	3	x	2 1/2	x	5/16	1.62	1.42	0.937	0.933	0.898	0.744	0.683
L 3x2.5x0.25	3	x	2 1/2	x	1/4	1.31	1.17	0.945	0.911	0.743	0.753	0.661

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

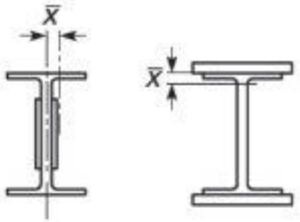
Case	Description of Element	Shear Lag Factor, *U*	Example
1	All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds. (except as in Cases 3, 4, 5 and 6)	$U = 1.0$	——
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds (Alternatively, for W, M, S and HP, Case 7 may be used.)	$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted 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	——

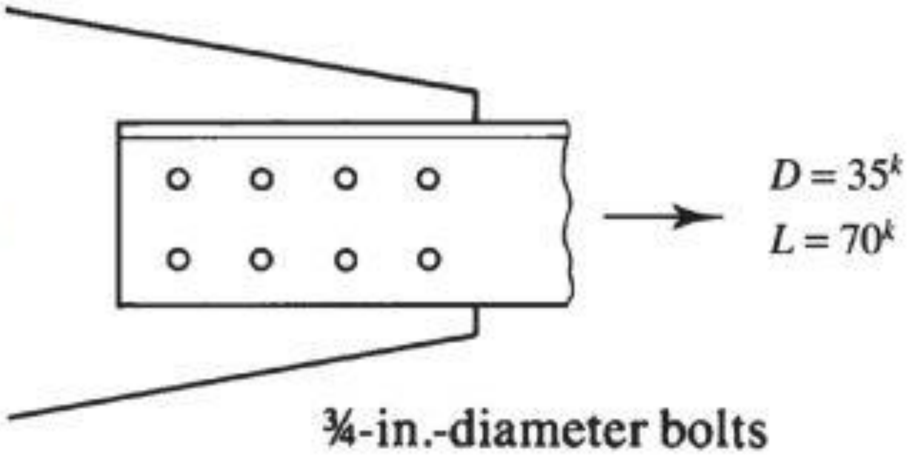
TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$	—
		with web connected with 4 or more fasteners in the direction of loading	$U = 0.70$	—
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	—
		with 2 or 3 fasteners per line in the direction of loading	$U = 0.60$	—

l = length of connection, in. (mm); w = plate width, in. (mm); $\bar{x}$ = connection eccentricity, in. (mm); B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)

Tension member design problem:

Select an unequal-leg angle tension member 15 ft long to resist a service dead load of 35 kips and a service live load of 70 kips. Use A36 steel (F_y = 36 ksi, F_u=58 ksi). The connection is shown in figure below. It shall be connected to a gusset plate using 8 nos. ¾-in dia. bolts in two rows as shown. Neglect block shear failure mode and follow LRFD principle.



Solution:

The factored load is

$P_u = 1.2D + 1.6L = 1.2(35) + 1.6(70)$
$= 154$ kips

On gross area: $P_u \leq \phi P_n = \phi F_y A_g$

$\therefore \; A_g = P_u/(\phi F_y) = 154/(0.9 \times 36)$
$= 4.75 \text{ in}^2.$

On net area: $P_u \leq \phi P_n = \phi F_u A_e$

$\therefore \; A_e = P_u/(\phi F_u) = 154/(0.75 \times 58)$
$= 3.54 \text{ in}^2.$

To find the net area A_n, we need U. However, at this stage, the section is unknown. We can assume U = 0.8 (Case 7, AISC Table D3.1, four bolts in a row).

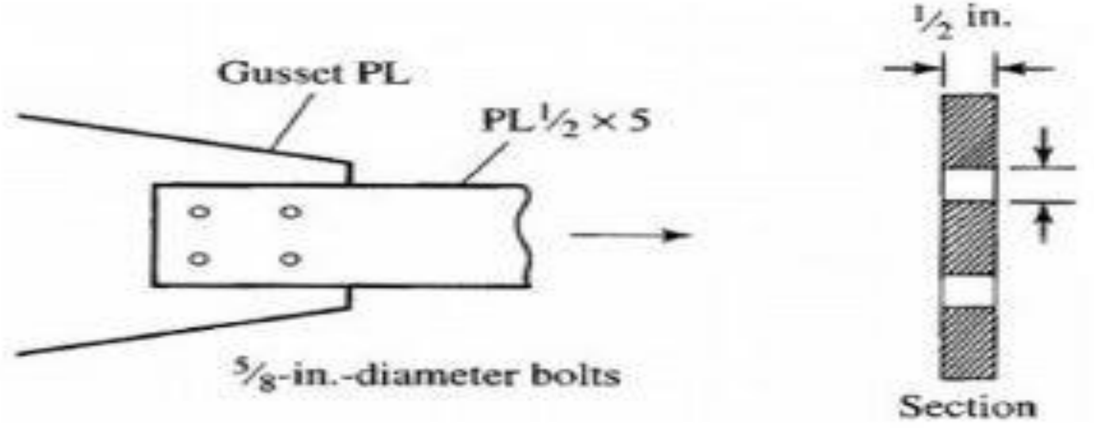
Thus, $A_n = A_e/U = 3.54/.8 = 4.425$ in^2.
$\therefore$ Gross area $A_g = A_n$ + bolt holes = $4.425 + 2(3/4 + 1/8)t$
Where t is the thickness of the angle which is unknown.
Conservatively, we choose t = ¾ in
$\therefore A_g = 4.425 + 2(3/4 + 1/8)(\ ¾\ ) = 5.74$ in^2 (governs)

Now we choose a section from AISC Manual with $A_g \geq 5.74$ in^2 and $t \leq$ ¾ in. We choose L6 x 4 x $^5/_8$
$A_g = 5.86$ in^2, $t = {}^5/_8$ in. $\therefore A_n = 5.86 - 2(3/4+1/8)\,{}^5/_8 = 4.77$ in^2.
$A_e = U A_n = 0.8(4.77) = 3.82$ in^2.

$\therefore$ Yield on gross area, $\phi P_n = \phi F_y A_g = 0.9(36)5.86 = 189.9$ kip,
Fracture on effective area, $\phi P_n = \phi F_u A_e = 0.75(58)3.82 = 166.2$ kip
Thus, tension capacity $\phi P_n = 166.2 > P_u$ OK.

Finding Capacity using LRFD (No staggered holes)

Ques. A PL $1/2 \times 5$ plate is connected by four $5/8$ inch diameter bolts. Determine tensile strength of the member. Assume A36 steel.



$$A_g = 5 \times \frac{1}{2} = 2.5 \text{ in}^2$$

$$A_n = A_g - A_h = 2.5 - 2\left[\frac{1}{2}\left(\frac{5}{8}+\frac{1}{8}\right)\right] = 1.75 \text{ in}^2$$

$$A_e = A_n U = 1.75 \times 1.0 = 1.75 \text{ in}^2 \quad [U = 1.0 \text{ for plate}]$$

Design strength based on yielding,

$$\phi_t P_n = 0.90 F_y A_g = 0.90 \times 36 \times 2.5 = 81 \text{ kip}$$

Design strength based on fracture,

$$\phi_t P_n = 0.75 F_u A_e = 0.75 \times 58 \times 1.75 = 76.1 \text{ kip}$$

Ans. 76.1 kip