Design Strength, $\varphi_t P_n$ of tension member shall be lower value obtained based on the following limit states of

1. Yielding in the gross section
2. Fracture in the effective net section.

1. Based on the Yielding in the gross section, design strength:

$$\varphi_t P_n = \varphi_t F_y A_g$$

Where
- Resistance factor, $\varphi_t = 0.90$
- Yield Stress = $F_y$ (ksi)
- Gross area of the member = $A_g$ (in$^2$)

2. Based on the Fracture on the Effective net section, design strength:

$$\varphi_t P_n = \varphi_t F_u A_e$$

Where
- Resistance factor, $\varphi_t = 0.75$
- Tensile strength = $F_u$ (ksi)
- Effective net area = $A_e$ (in$^2$)

Effective net area = Shear Slag Coefficient X Net Area

$$A_e = U \times A_n$$

Where
- $U = 1 - [x / L]$
Example 1:
(a) Determine the tension design strength (LRFD) of gusset plate and an angle connection. Use A36 Steel.
(b) The applied tension loads are 30 kips dead load (D) and 40 kips live load (L). Determine whether the connection is adequate considering tension only.

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AISC Steel Manual Table 1-7 (P 1-42),

\[ x = 1.18 \text{ in.} \]
\[ Ag = 3.75 \text{ in}^2 \]
\[ An = Ag - (\text{Bolt Dia} + 1/8')(\text{Angle thickness}) = 3.75 - (3/4 + 1/8)(1/2) = 3.31 \text{ in}^2 \]

\[ U = 1 - (1.18/6) = 0.8 \]
\[ Ae = 0.8 (3.31) = 2.65 \text{ in}^2 \]

AISC Table 2-5 (P2-41), for A36 steel, \( F_y = 36 \text{ ksi}, F_u = 58 \text{ ksi} \)
For the limit state of yielding, Design tensile strength,

\[ \phi_t P_n = \phi_t F_y A_g \]

\[ = (0.9)(36)(3.75) = 121.5 \text{ kips} \]

For the limit state of rupture, Design tensile strength,

\[ \phi_t P_n = \phi_t F_u A_e \]

\[ = (0.75)(58)(2.65) = 116 \text{ kips (Controls)} \]

Therefore, limit state of rupture controls, and design tensile strength = 116 kips

(b) \[ Pu = 1.4 D = 1.4(30) = 42 \text{ kips} < 116 \text{ kips OK} \]

\[ Pu = 1.2 D + 1.6 L = 1.2(30) + 1.6(40) = 100 \text{ kips} < 116 \text{ kips OK} \]


**BLOCK SHEAR**

Design Block shear strength = $\varphi R_n$ where $\varphi = 0.75$

$$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$$

Where

$A_{gv}$ = gross area in shear

$A_{nv}$ = net area in shear

$A_{nt}$ = net area in tension

$U_{bs}$ = 1.0 for uniform tension stress, and 0.5 for non-uniform tension stress.

For tension member, the tensile stress is assumed to be uniform. For tension member, use $U_{bs} = 1.0$
Example 2: Determine the design block shear strength of the gusset plate.
Given:
Gusset plate thickness = ½\"; A36 steel
Bolts dia= 7/8\".

**BLOCK SHEAR FAILURE OF A PLATE**
Design Block Shear Strength = \( \phi R_n \) where \( \phi = 0.75 \)

\[
R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}
\]

Where

\( A_{gv} \) = gross area in shear = \( 2(3+3+2)(0.5) = 8 \text{ sq.in} \)
\( A_{nv} \) = net area in shear = \( 2(8 - 2.5(7/8+1/8))(0.5) = 5.5 \text{ sq.in} \)
\( A_{nt} \) = net area in tension = \( (6 - (7/8 +1/8))(0.5) = 2.5 \text{ sq.in} \)

\( U_{bs} = 1.0 \) for uniform tension stress.

\[
R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt}
\]
\[
= 0.6(58)(5.5) + 1.0(58)(2.5) = 191.4 + 145 = 336.4 \text{ kips}
\]

But not greater than

\[
R_n = 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}
\]
\[
= 0.6(36)(8) + 1.0(58)(2.5) = 172.8 + 145 = 317.8 \text{ kips}
\]

(GOVERNS)

Selecting the lowest nominal strength = 317.8 kips

Design block shear strength = \( \phi R_n = 0.75(317.8) = 238.35 \text{ kips} \)
Example 3:

Given:
Wide Flange W Section : W14 X 43 A992
Splice Plate A : ½" Thick
Bolts : 7/8" dia.
LRFD available strength of a group of six bolts is 211 kips.

Note: The Splice plates will be selected so that they do not limit the member strength.
Determine the design strength of the splice between two W-shapes.

Solution:

W14 X43  A992 (Fy = 50 ksi)

Ag = 12.6 sq. in
tf = 0.53 in

1. For the limit state of yielding, Design tensile strength,

$$\phi_t P_n = \phi_t F_y A_g$$

$$= (0.9)(50)(12.6) = 567 \text{kips}$$

2. Net Area

Area to be deducted from each flange = 2(7/8 +1/8)(0.530) = 1.06 sq.in

Net Area, An = 12.6 – 2(1.06) = 10.5 sq.in

3. Shear Lag Factor, U

W14X43 is treated as two Tee sections, each WT 7X21.5
AISC Table 1-8, x = 1.31, and L=6

$$U = 1 - (1.31/6) = 0.782$$

AISC Table D3.1 (Page 16.1-29), with bf< 2/3d, U=0.85

4. For the limit state of rupture, Design tensile strength,

$$Ae = U An = 0.85 (10.5) = 8.925 \text{sq.in}$$

$$\phi_t P_n = \phi_t F_u A_e$$

$$= (0.75)(65 \text{ksi})(8.925) = 435 \text{kips}$$
5. Block Shear strength of the flanges

Design Block Shear Strength = $\varphi R_n$, where $\varphi = 0.75$

$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

Where

$A_{gv} = \text{gross area in shear} = 4(3+3+2)(0.53) = 16.96 \text{ sq.in}$

$A_{nv} = \text{net area in shear} = 16.96 - 4[2.5(7/8+1/8)(0.53)] = 16.96 - 5.3 = 11.66 \text{ sq.in}$

$A_{nt} = \text{net area in tension} = 4 [2 - 0.5(7/8+1/8)](0.53) = 3.18 \text{ sq.in}$

$U_{bs} = 1.0$ for uniform tension stress.

$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt}$

$= 0.6(65)(11.66) + 1.0(65)(3.18) = 454.74 + 206.7 = 661.44 \text{ kips}$

$R_n = 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

$= 0.6(50)(16.96) + 1.0(65)(3.18) = 508.8 + 206.7 = 715.5 \text{ kips}$

Therefore, $\varphi R_n = 0.75(661.44) = 496 \text{ kips}$

6. Compare the design strength for each limit states:

Bolt design strength = 211 x 2 = **422 kips** (CONTROLS)

Yield of the member = 567 kips

Rupture of the member = 435 kips

Block Shear for the member = 496 kips

Therefore, design strength of the tension splice is **422 kips**