University of California, San Diego
Faculty of Engineering

DESIGN OF TENSION MEMBERS

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INTRODUCTION

Tension Members

'Tension members are axially loaded members stressed in tension and are used in steel structures in various forms. They are used in trusses as web and chord members, hanger and sag rods, diagonal bracing for lateral stability, and lap splices such as in a moment connection.

Beams and columns are subjected to compression buckling (such as lateral-torsional buckling, Euler buckling, and local buckling) and must be checked for this failure mode, but tension members are not subjected to the same lateral instability since compression stresses do not exist.

The exception to this is the special case when the applied tension load is eccentric to the member in question, inducing an applied moment and therefore creating the possibility of lateral instability.'

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Examples of Tension Members

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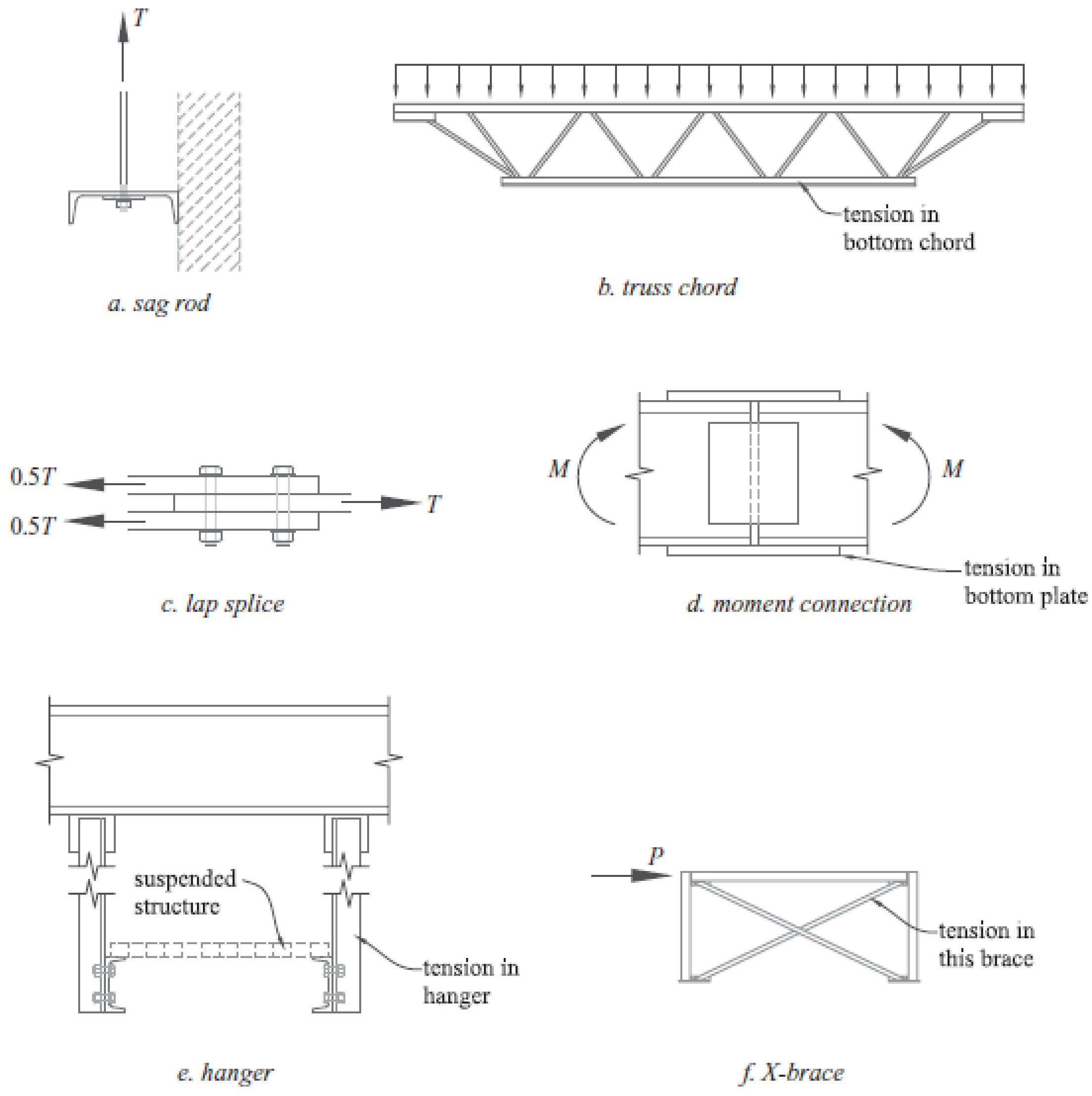


(Abi Aghayere and Jason Vigil (2015))

INTRODUCTION

Design Criteria

'The basic design check for a tension member is to provide enough cross-sectional area to resist the applied tensile force.

In practice, however, pure tension members do not typically exist in this form and several additional factors must be considered. One common example is tension members with nonuniform cross sections, such as the case when a tension member is connected with bolts at the ends. Eccentric loading must also be considered, such as a single steel angle with a concentric load connected to a gusset plate.'

INTRODUCTION

Slenderness

'Even though slenderness is not a direct design concern, the AISC specification does recommend an upper limit on the slenderness ratio L/i for tension members. This upper L/i limit is equal to 300 for tension members and 200 for compression members, where L is the length of the member and i is the radius of gyration.

The recommendation does not apply to rods or hangers in tension and is not absolutely required for tension members. '

INTRODUCTION

Slenderness

'There is no slenderness for design of tension members. However, too long tension members may deflect excessively due to their own weight and vibrate when subject to wind forces. To reduce the excessive deflection and vibration problems in tension members, the following stiffness criteria is suggested (ANSI/AISC 360-10, D1):'

$$\frac{L}{i} \leq 300 \qquad i = radius\ of\ \ gyration = \sqrt{\frac{I}{A}}$$

I = moment of inertia of the cross secton

A = gross area of the cross section

ANALYSIS OF TENSION MEMBERS

Failure Modes (Sınır Durumları)

- Tensile yielding (ÇYTYE Bölüm 7)
- Tensile rupture (ÇYTYE Bölüm 5.4.3 ve 7)
- Block shear (Blok kesme) (ÇYTYE Bölüm 13.4.3)

Tensile yielding occurs when the stress on the gross area of the section is large enough to cause excessive deformation.

Tensile rupture occurs when the stress on the effective area of the section is large enough to cause the member to fracture, which usually occurs across a line of bolts where the tension member is weakest.

ANALYSIS OF TENSION MEMBERS

Failure Modes (Sınır Durumları)

- Tensile yielding (ÇYTYE Bölüm 7)
- Tensile rupture (ÇYTYE Bölüm 5.4.3 ve 7)
- Block shear (Blok kesme) (ÇYTYE Bölüm 13.4.3)

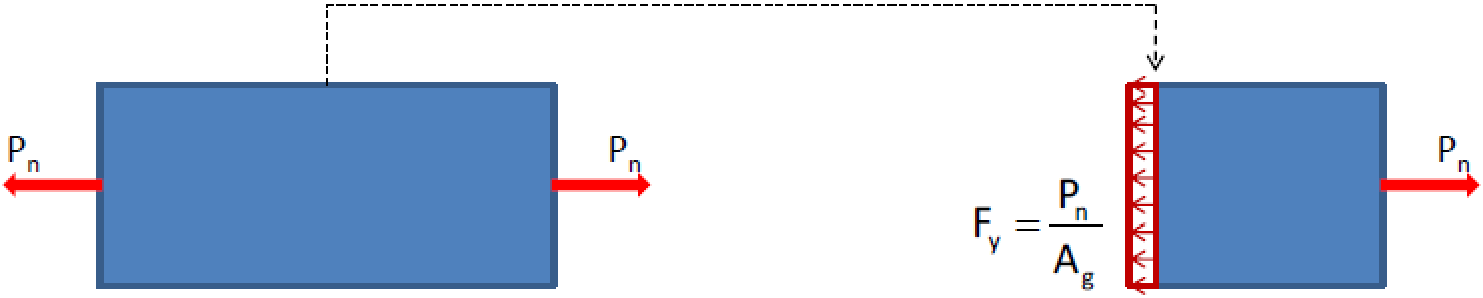
Block shear is a tearing failure mode (rupture) that occur along the perimeter of the bolt holes.

Block shear mode combines tensile failure on one plane and shear failure on a perpendicular plane. The failure path is defined by the centerlines of the bolt holes.

ANALYSIS OF TENSION MEMBERS

1) Tensile Yielding (Bölüm 7)

This limit state defines the yielding of the gross section of a tension member without.



$$P_n = T_n = F_y A_g \qquad \phi = 0.90$$

$$\Omega = 1.67$$

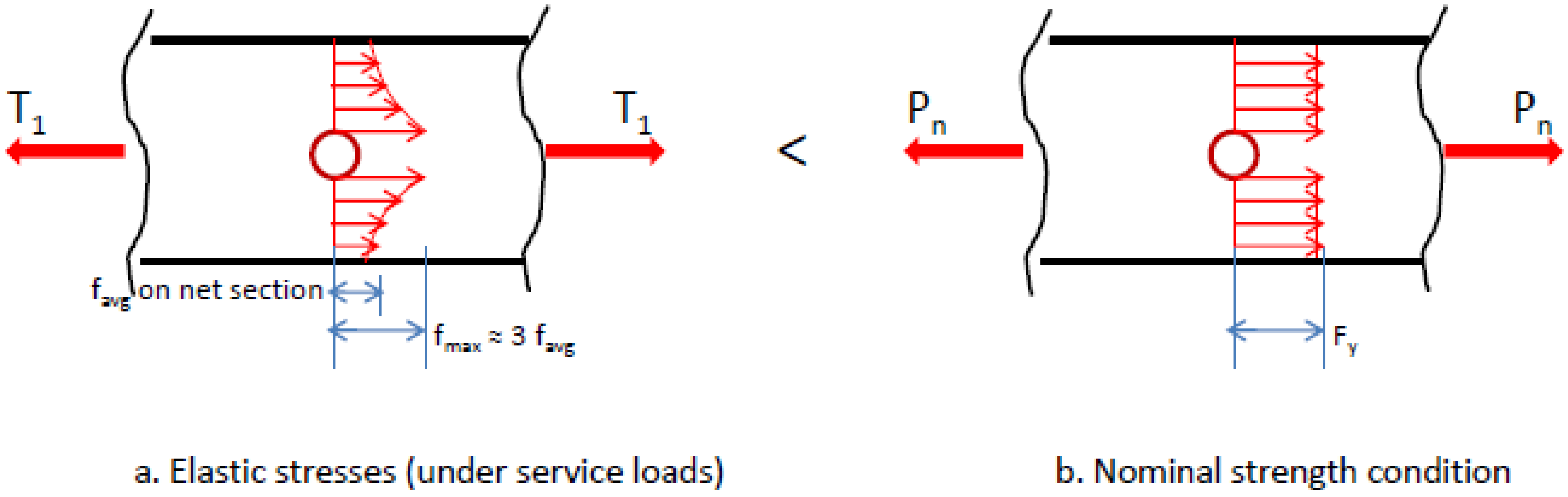
T_n = nominal tensile strength (N)

F_y = yield stress (MPa)

A_g = gross-sectional area (mm^2)

2) Tensile Rupture (Bölüm 5.4.3)

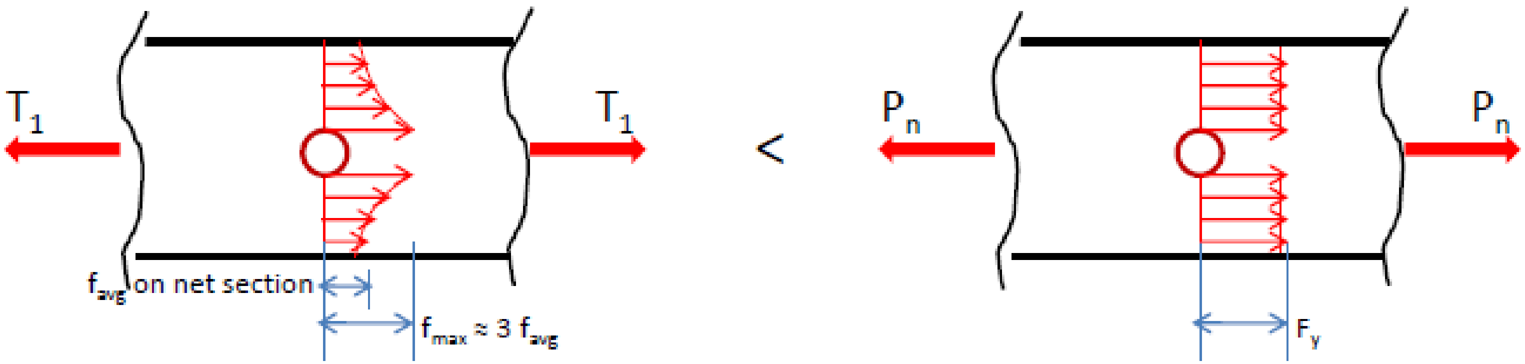
This limit state is defined as the fracture of the effective net area.



Stress distribution with holes present

2) Tensile Rupture (Bölüm 5.4.3)

This limit state is defined as the fracture of the effective net area.



a. Elastic stresses (under service loads) b. Nominal strength condition

Stress distribution with holes present

T_n = nominal tensile strength (N)

F_y = yield stress (MPa)

A_g = gross-sectional area (mm^2)

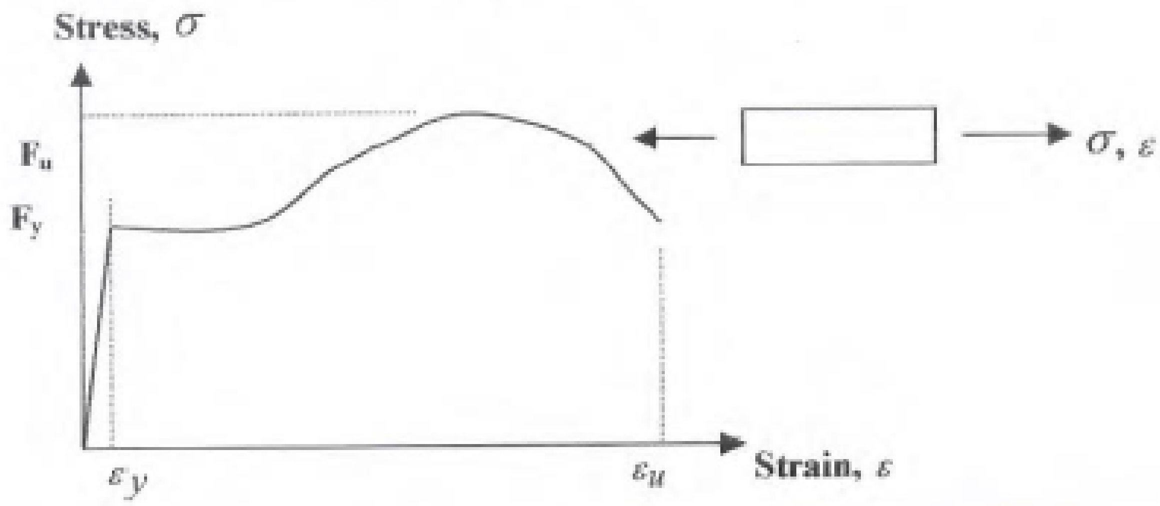
$$P_n = T_n = F_y A_g$$

$$\phi = 0.75$$

$$\Omega = 2.0$$

2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

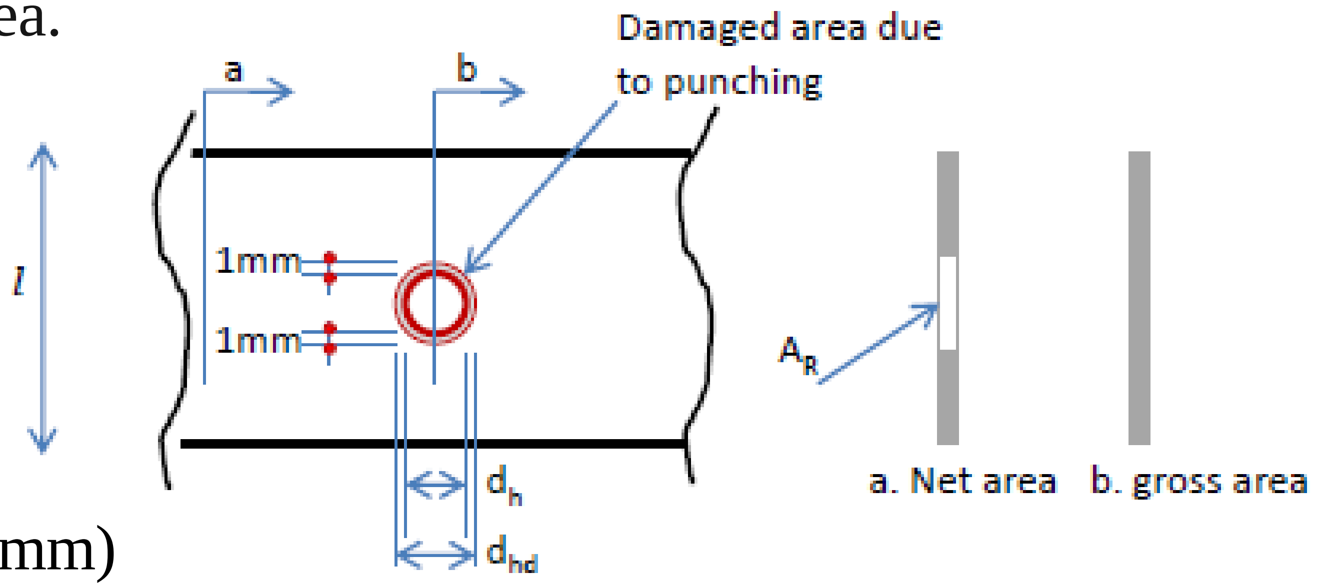
$$T_n = F_u A_e$$



T_n = nominal tensile strength (N)
F_u = specified minimum tensile strength (MPa)
$A_e = UA_n$ = effective net area (mm^2)
A_n = net area (mm^2)
U = reduction coefficient (an efficiency factor)

2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

Definition of net area.



d_b = bolt diameter (mm)

d_h = hole diameter (mm) = $d_b + 2^{mm}$ (for standard holes)

d_{hd} = total width to be deducted from the width (l) of the cross-section for the computation of net area (for tension and shear) (mm) = $d_h + 2^{mm}$ (for every hole)

$A_n = A_g - d_{hd}t$

t = thickness (mm)

ANALYSIS OF TENSION MEMBERS

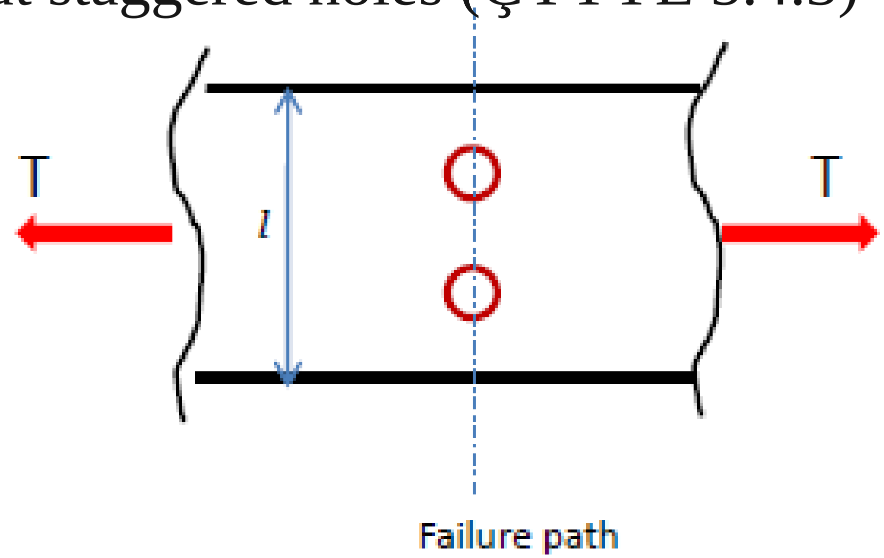
2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

TABLO 13.8 – KARAKTERİSTİK DELİK BOYUTLARI, (mm)

Bulon	Delik Boyutları			
	Standart Dairesel Delik Çapları	Büyük Dairesel Delik Çapları	Kısa Oval Delik (Genişlik × Uzunluk)	Uzun Oval Delik (Genişlik × Uzunluk)
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	26	30	26 × 32	26 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75
≥ M36	$d + 3$	$d + 8$	$(d + 3) \times (d + 10)$	$(d + 3) \times 2.5d$

2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

Net area without staggered holes (ÇYTYE 5.4.3)



$A_n = A_g - n d_{hd} t$

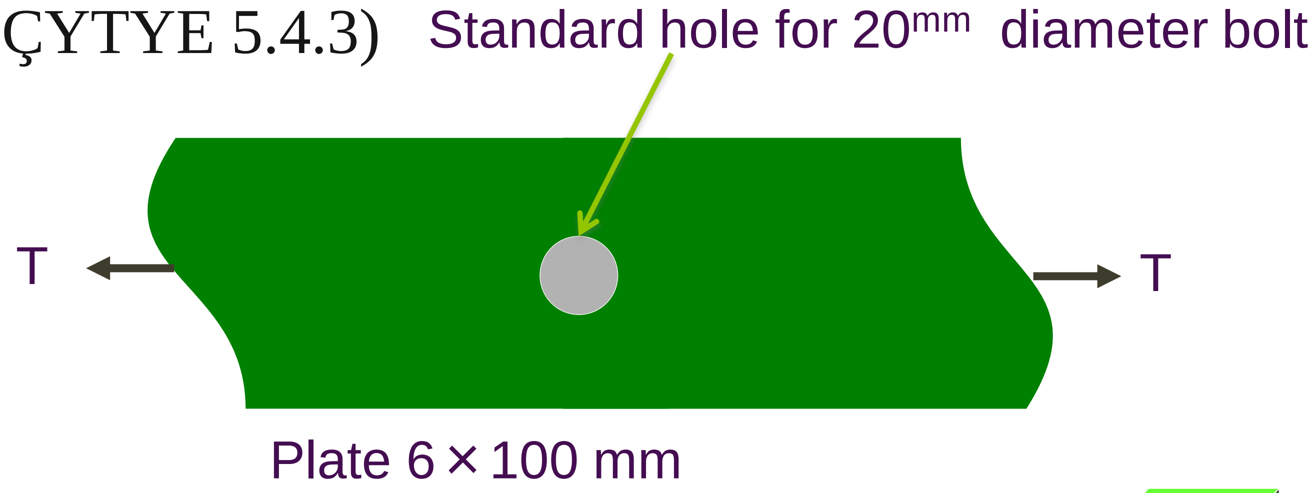
n = number of holes transverse to the direction of holes

$d_{hd} = d_h + 2^{mm}$ (for every hole)

t = thickness of the plate (mm)

2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

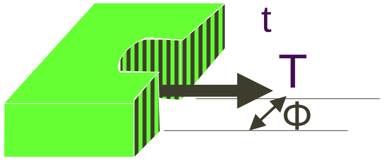
Example: (ÇYTYE 5.4.3)



$$A_g = 100^{mm} x 6^{mm} = 600\ mm^2$$

$$d_h = 22^{mm}\ (Tablo 13.8)$$

$$d_{hd} = 20^{mm} + 2^{mm} + 2^{mm} = 24\ \text{mm}$$

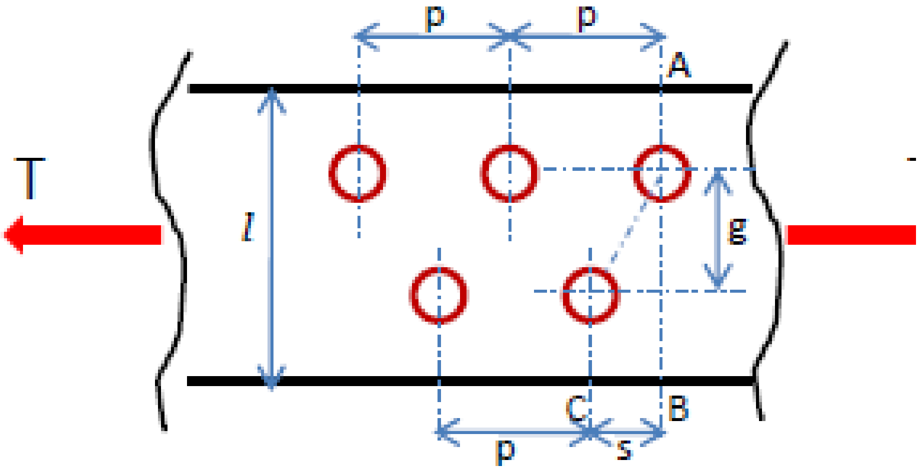


$$A_n = A_g - (d_{hd})(t)$$

$$= 600^{mm2} - 24^{mm}(6^{mm}) = 456\ mm^2$$

2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

Net area with staggered holes (ÇYTYE 5.4.3)



$$l_n = \text{smaller of} \begin{cases} l - \Sigma(n.d_{hd}) & (\text{along } A-B) \\ l - \Sigma(n.d_{hd}) + \Sigma(\dfrac{s^2}{4g}) & (\text{along } A-C) \end{cases}$$

accounts for the increase in strength due to the added cross-sectional area at an angle in the failure plane

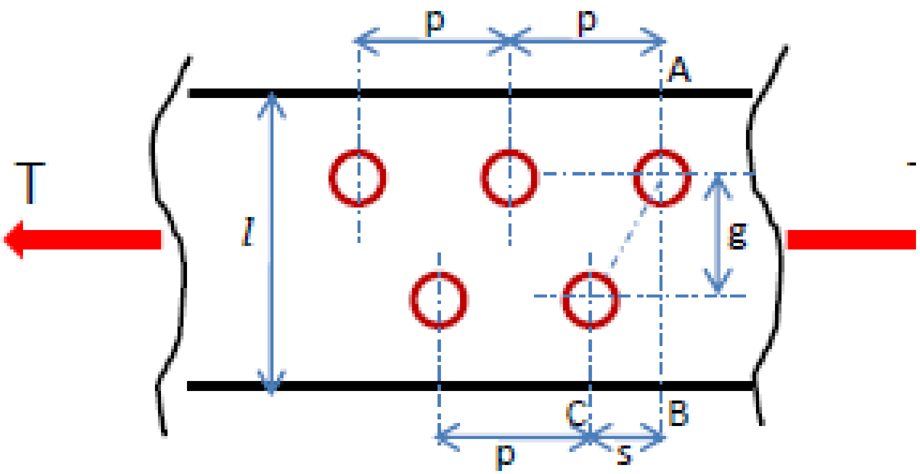
$A_n = l_n t$

s = stagger or spacing of adjacent holes parallel to the loading direction (mm)

g = gage distance transverse to the loading direction (mm)

2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

Net area with staggered holes (ÇYTYE 5.4.3)



$$l_n = smaller of \begin{cases} l - \Sigma(n.d_{hd}) & (along A-B) \\ l - \Sigma(n.d_{hd}) + \Sigma(\frac{s^2}{4g}) & (along A-C) \end{cases}$$

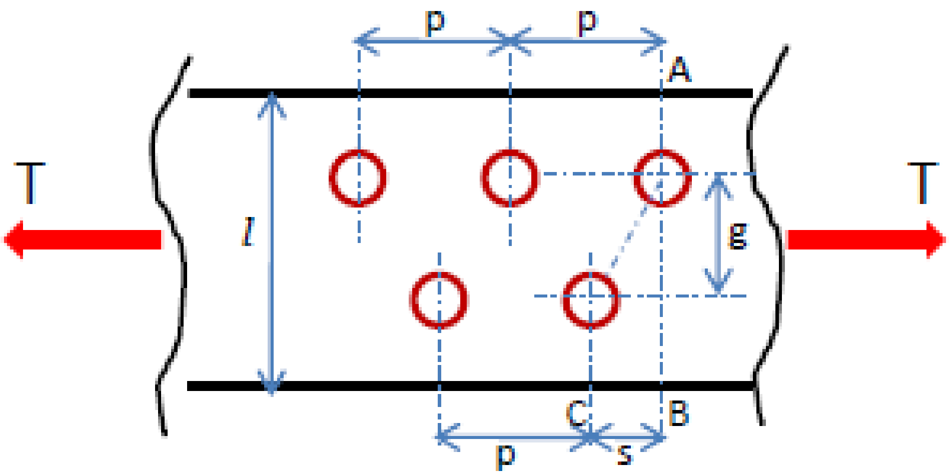
accounts for the increase in strength due to the added cross-sectional area at an angle in the failure plane

The controlling failure line is the one that gives the largest stress on an effective net area. In many cases, the critical failure path is also the path that has the minimum net area.

In determining the net area across plug or slot welds, the weld metal will not be considered as adding to the net area.

2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

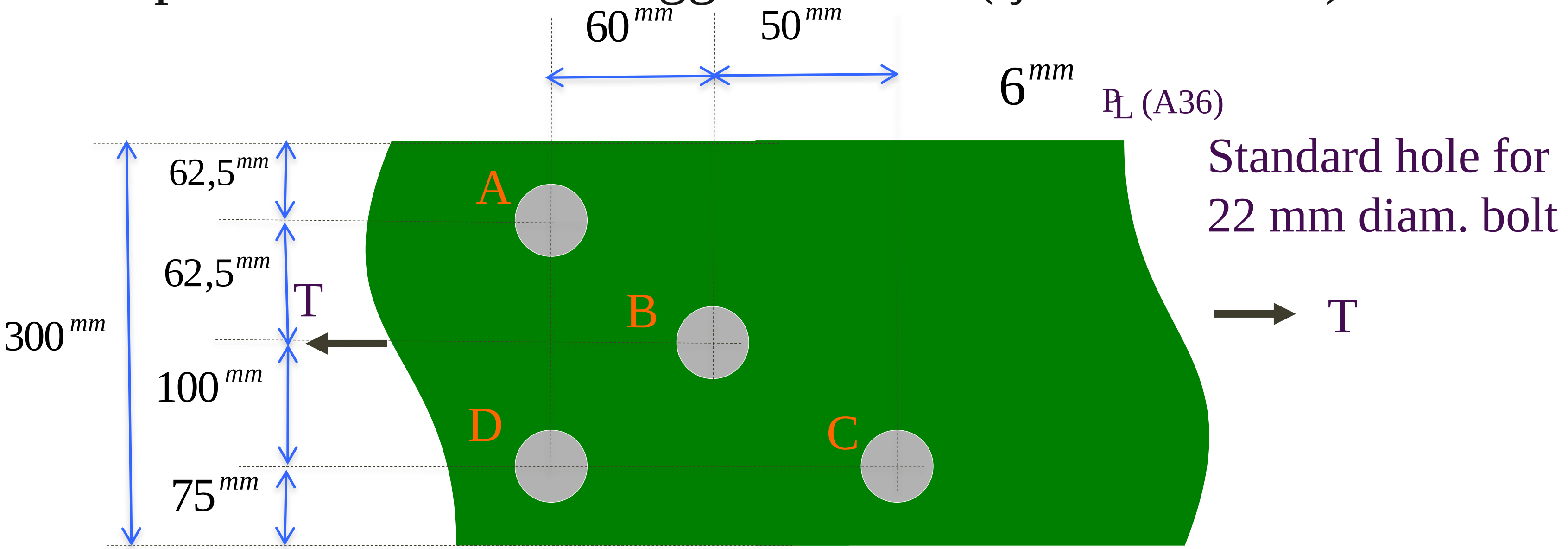
Net area with staggered holes (ÇYTYE 5.4.3)



$$A_n = A_g - \sum d_{hd} t + \sum \frac{s^2 t}{4g} (Denk.5.3)$$

2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

Example: Net area with staggered holes (ÇYTYE 5.4.3)

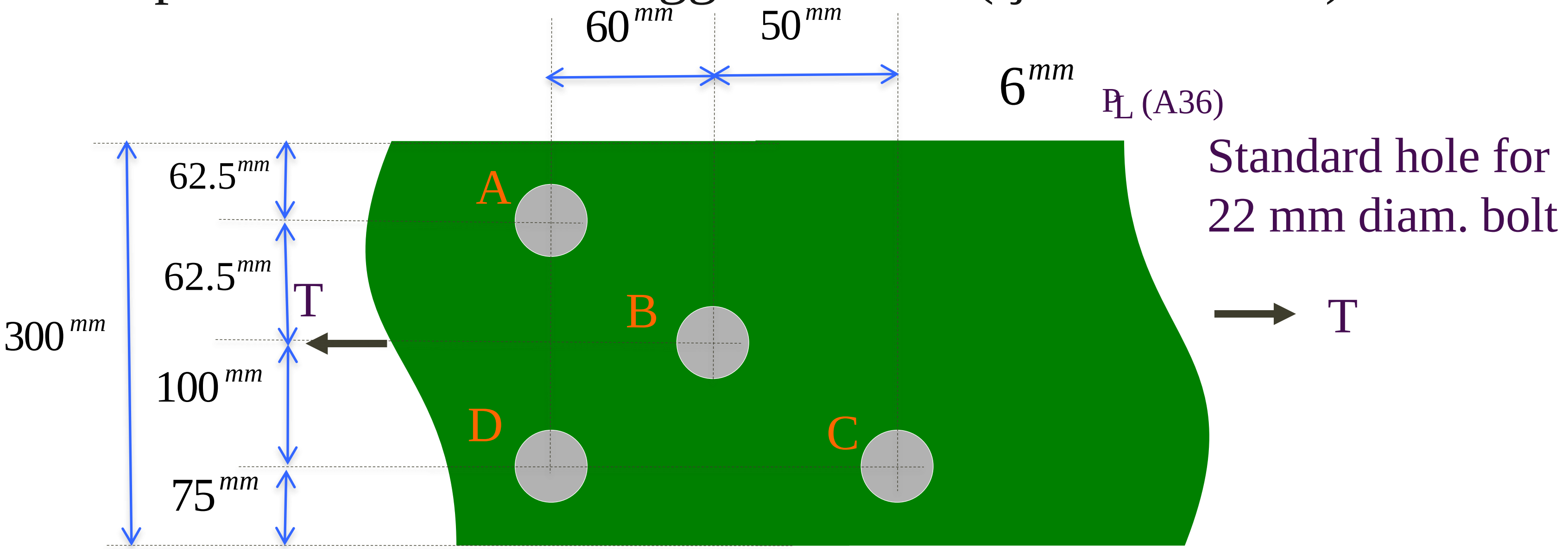


Following AISC-B4.3B or ÇYTYE 5.4.3

Path AD (two holes)□ $\left[300^{mm} - 2\left(22^{mm} + 2^{mm} + 2^{mm}\right)\right]x6^{mm} = 1488\ \text{mm}^2$

2) Tensile Rupture (Bölüm 5) (Fracture of the effective net area)

Example: Net area with staggered holes (ÇYTYE 5.4.3)



Path ABD (three holes; two staggers)

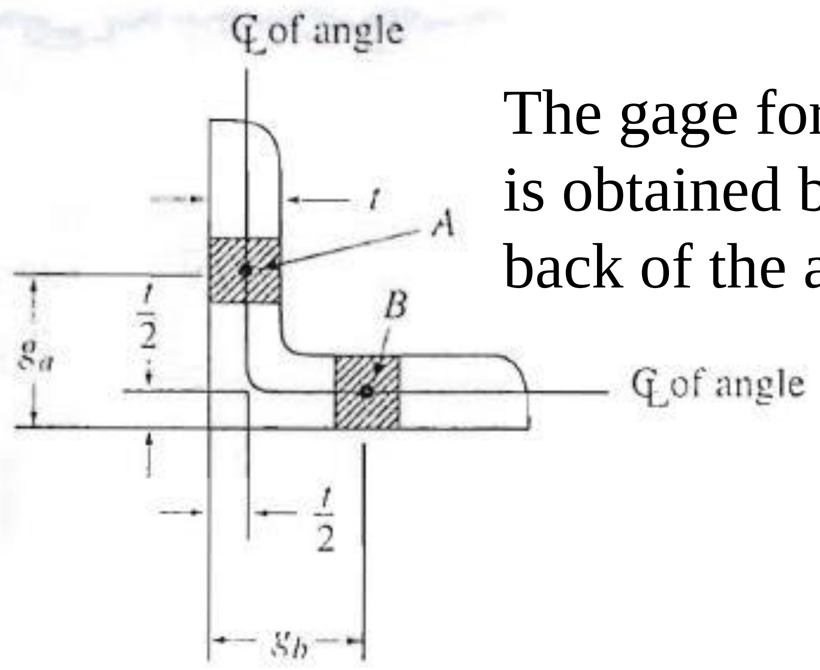
$$\left[300^{mm} - 3\left(22^{mm} + 2^{mm} + 2^{mm}\right) + \frac{\left(60^{mm}\right)^2}{4\left(62.5^{mm}\right)} + \frac{\left(60^{mm}\right)^2}{4\left(100^{mm}\right)}\right] x6^{mm} = 1472.4\ \text{mm}^2$$

Path ABC (three holes; two staggers)

$$\left[300^{mm} - 3\left(22^{mm} + 2^{mm} + 2^{mm}\right) + \frac{\left(60^{mm}\right)^2}{4\left(62.5\right)^{mm}} + \frac{\left(50^{mm}\right)^2}{4\left(100^{mm}\right)}\right] x6^{mm} = \boxed{1456\ \text{mm}^2}$$

2) Tensile Rupture (Bölüm 5.4.3)

Net area with staggered holes in angles



The gage for holes in opposite adjacent legs is obtained by the sum of the gages from the back of the angles less the thickness

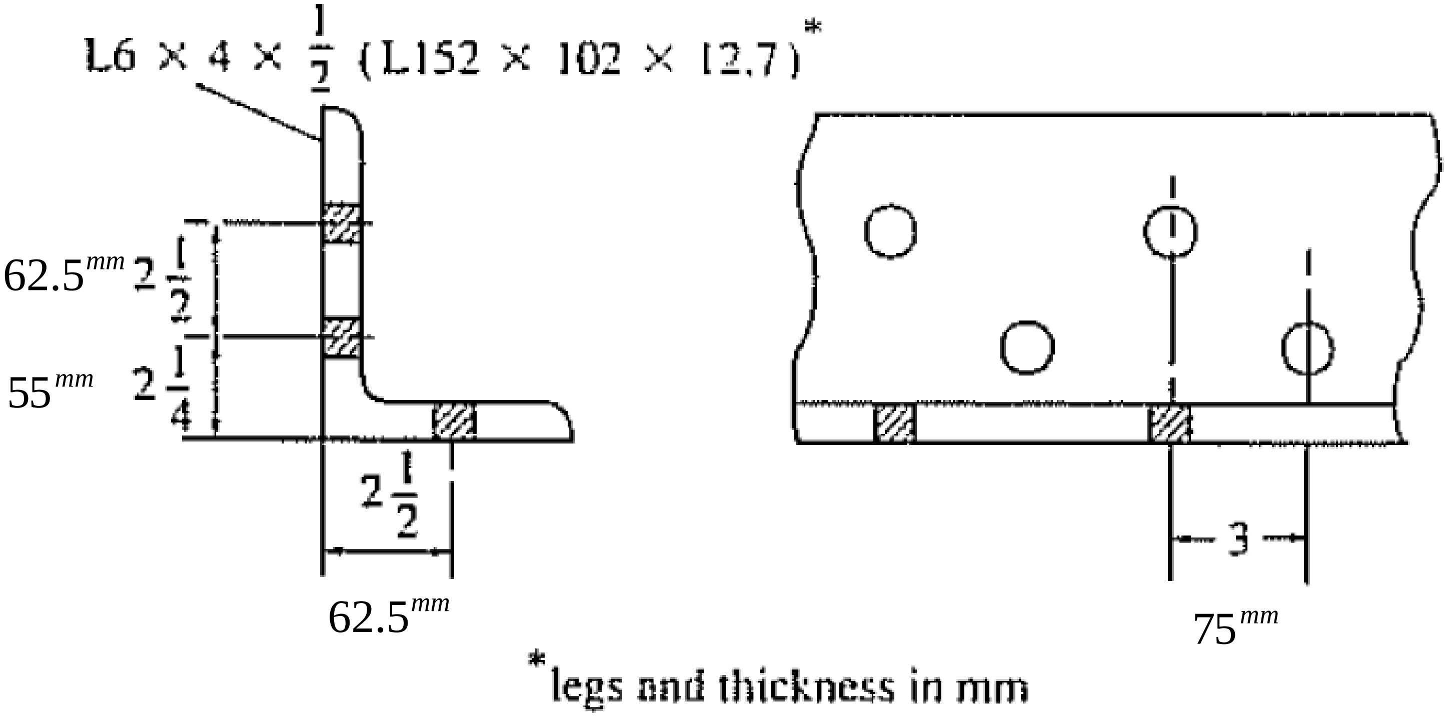
Figure Gage dimensions for an angle.

$g = g_A + g_B - t$: The distance from A to B

2) Tensile Rupture (Bölüm 5.4.3)

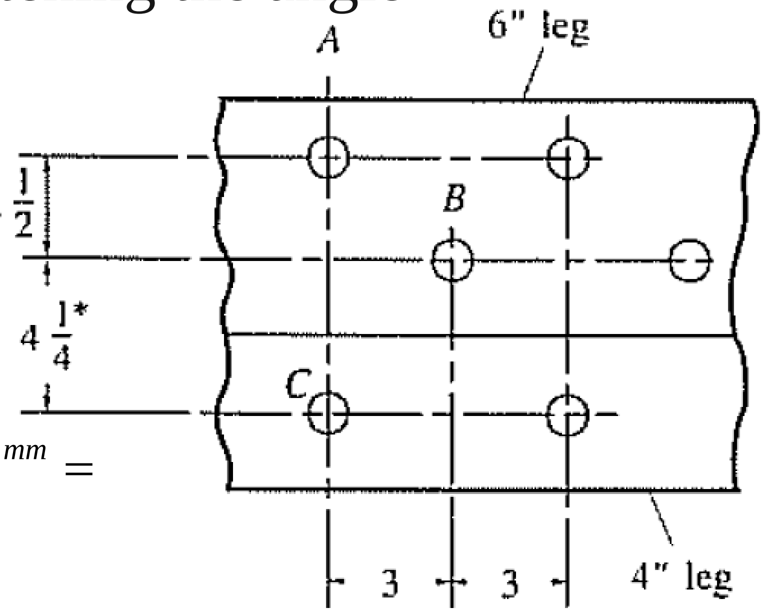
Example: Determine the net area of the angle

Standard hole for M22 bolt: 22 mm diameter



2) Tensile Rupture (Bölüm 5.4.3)

Example: Flattening the angle



62.5^{mm}

108^{mm}

$$g_1 + g_2 - t =$$

$$55^{mm} + 62.5^{mm} - 12.7^{mm} =$$

$$104.8\,mm$$

$$A_n = A_g - \sum Dt + \sum \frac{s^2}{4g} t$$

Path AC (two holes)➔ $\left[\left(152.5^{mm} + 102^{mm} - 12.7^{mm}\right) - 2\left(22^{mm} + 2^{mm} + 2^{mm}\right)\right] x 12.7^{mm} = 2410.5\,mm^2$

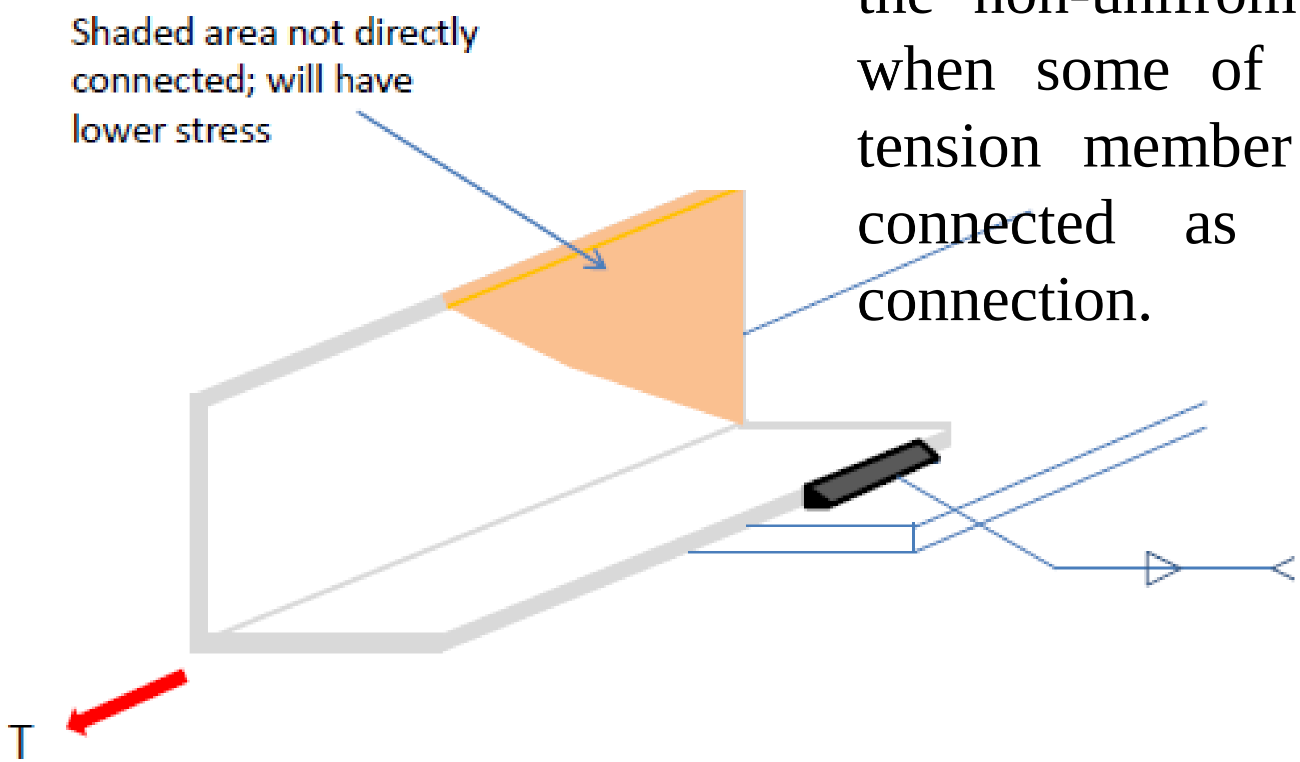
Path ABC ➔ $\left(\left(152.5^{mm} + 102^{mm} - 12.7^{mm}\right) - 3\left(22^{mm} + 2^{mm} + 2^{mm}\right) + \left[\frac{\left(75^{mm}\right)^2}{4\left(62.5^{mm}\right)} + \frac{\left(75^{mm}\right)^2}{4\left(104.8^{mm}\right)}\right]\right) x 12.7^{mm}$

$$= 2531\,\text{mm}^2$$

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

The shearlag factor, U, accounts for the non-unifrom stress distribution when some of the elements of a tension member are not directly connected as in the angle-bar connection.



2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

- ÇYTYE 7.1.3 requires that *effective net area* A_e

$$A_e = UA_n$$

U = gerilme düzensizliği etki katsayısı

A_n = eleman net enkesit alanı

- ÇYTYE Table 7.1 Case 2 (except plates and HSS (hollow structural sec.))

$$U = 1 - \frac{\bar{x}}{l} \leq 0.9$$

$\bar{x}$ = distance from centroid of element being connected eccentrically to plane of load transfer

l = length of connection in the direction of loading

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

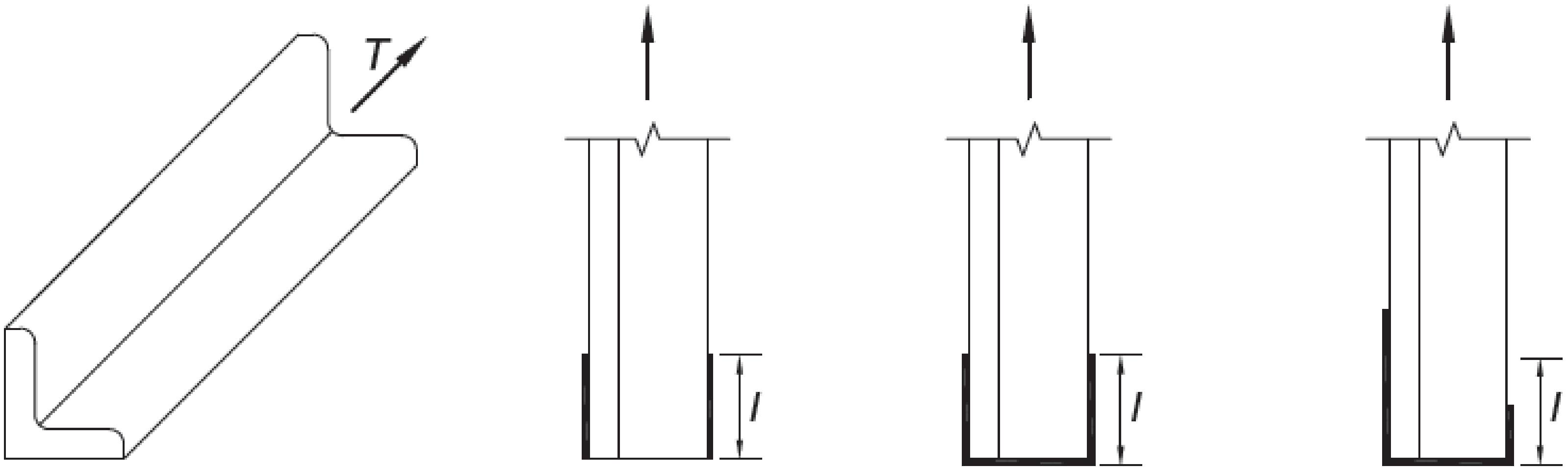
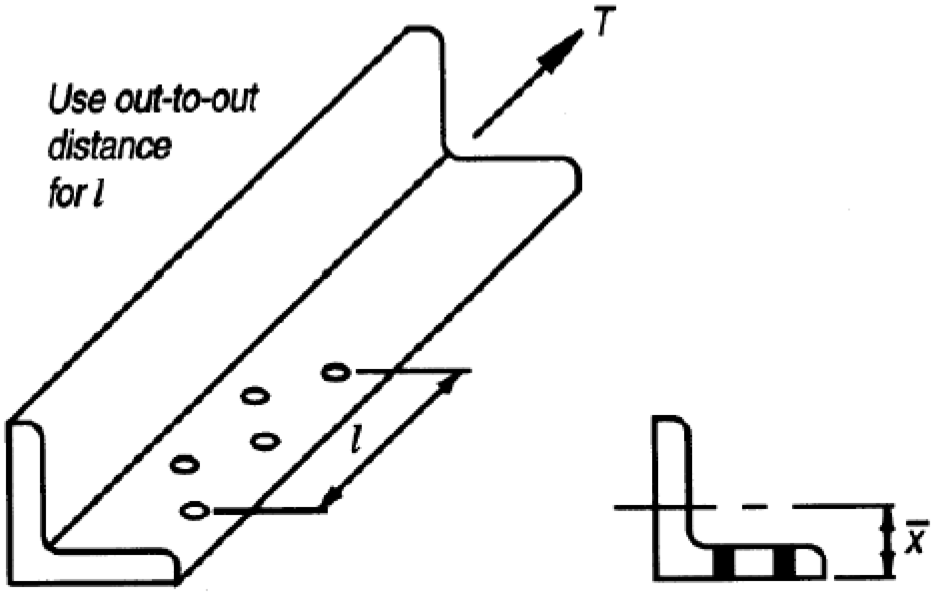


Fig. C-D3.4. Determination of l for calculation of U for connections with longitudinal and transverse welds.

2) Tensile Rupture (Bölüm 7)

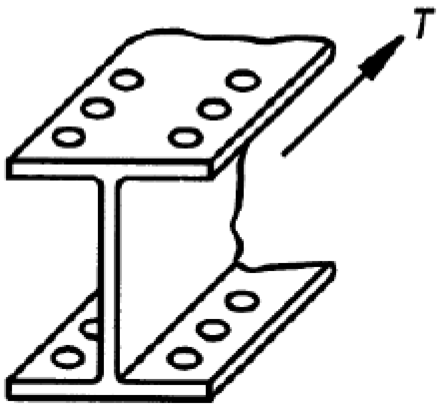
Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

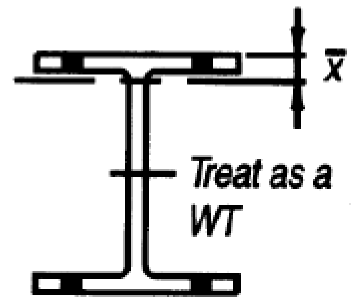


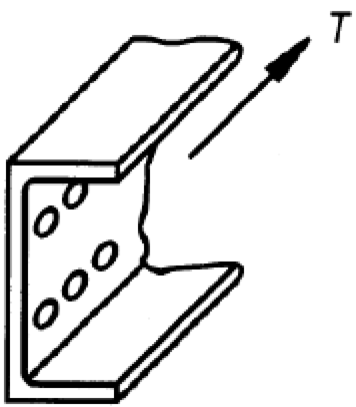
AISC 360: C-D3.4

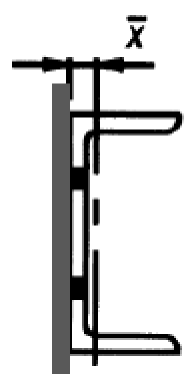
2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$







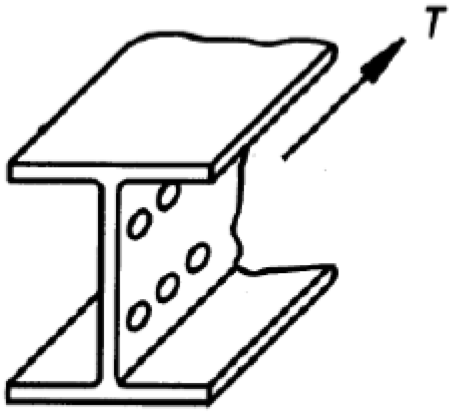


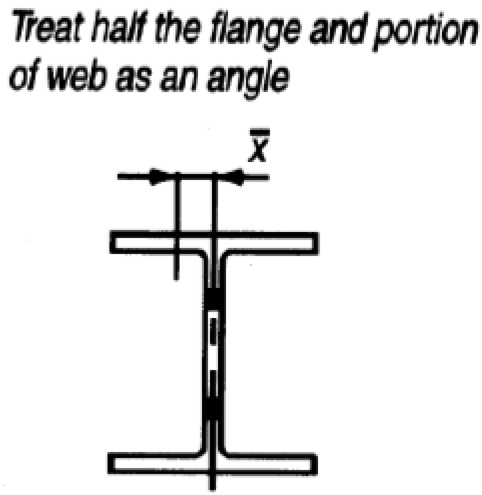
AISC 360: C-D3.4

ANALYSIS OF TENSION MEMBERS

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$





AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

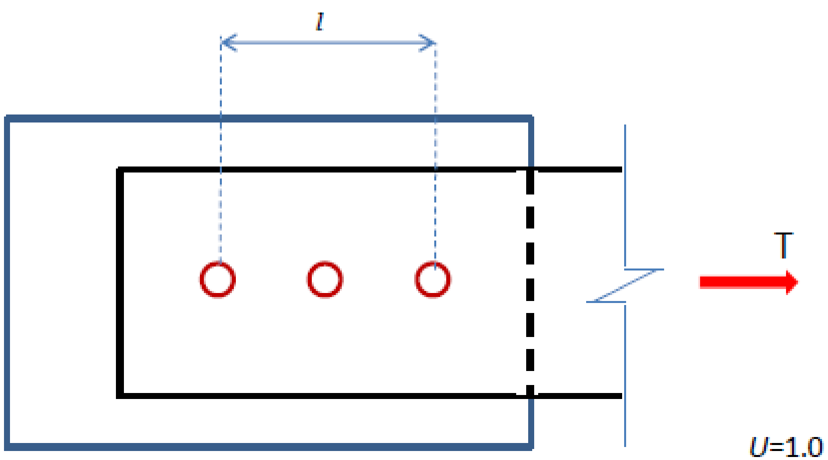


Plate – all bolted

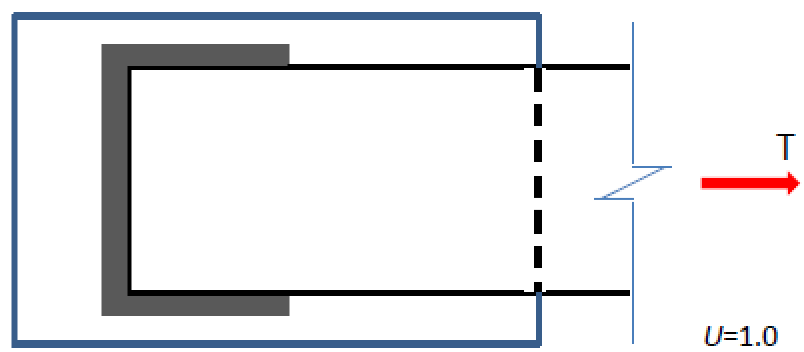


Plate – all welded

AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

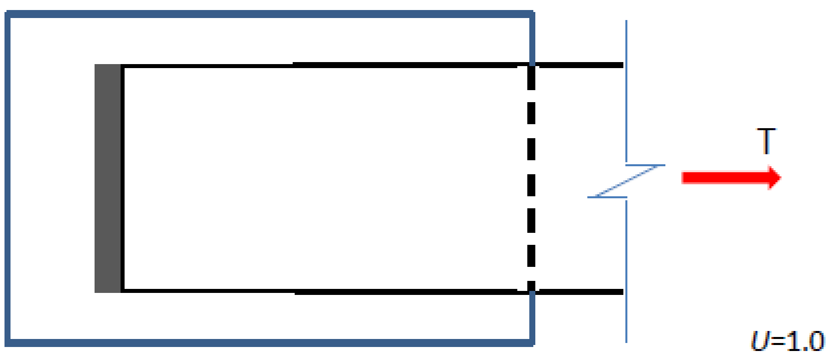


Plate – transverse weld

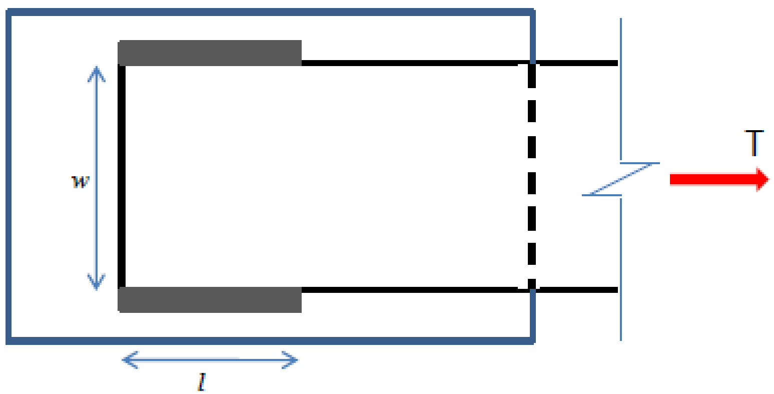


Plate – longitudinal welds

$l \geq 2w, \quad U=1.0$

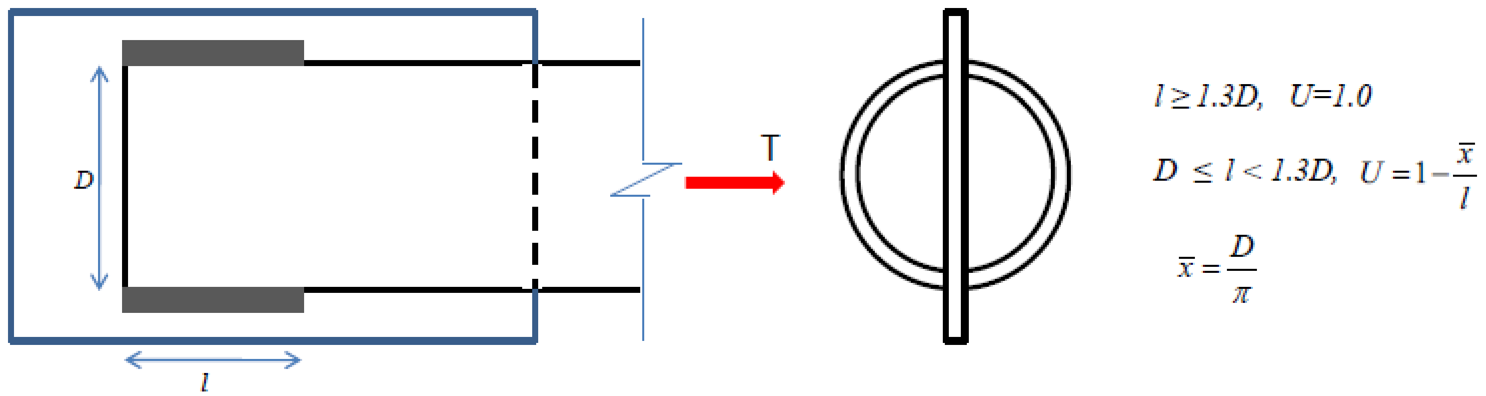
$1.5w \leq l < 2w, \quad U=0.87$

$w \leq l < 1.5w, \quad U=0.75$

AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

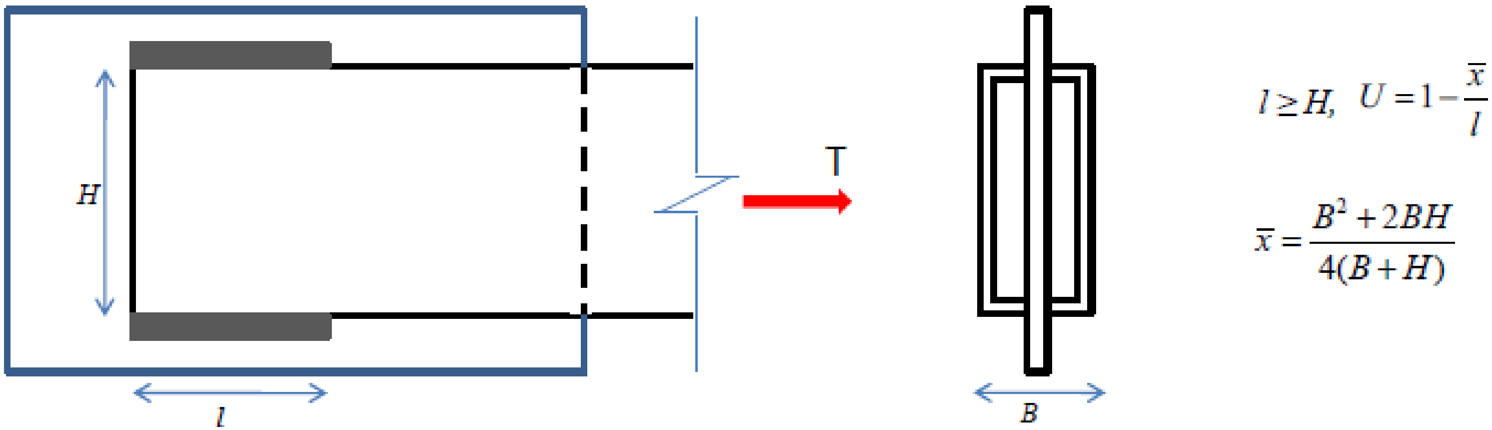


Round HSS – single concentric gusset plate

AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

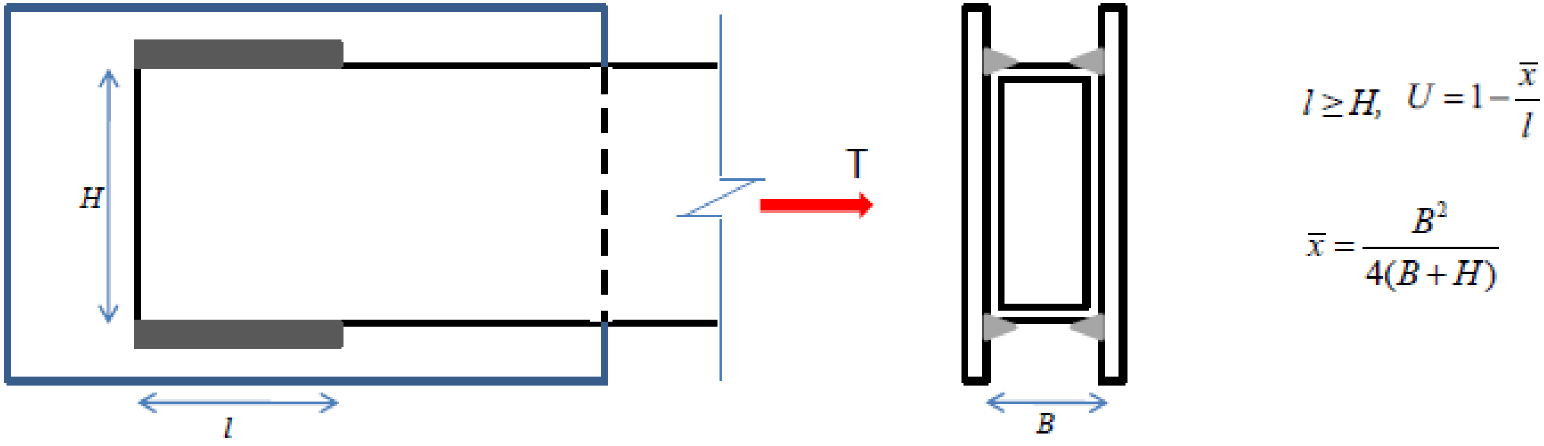


Rectangular HSS – single concentric gusset plate

AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

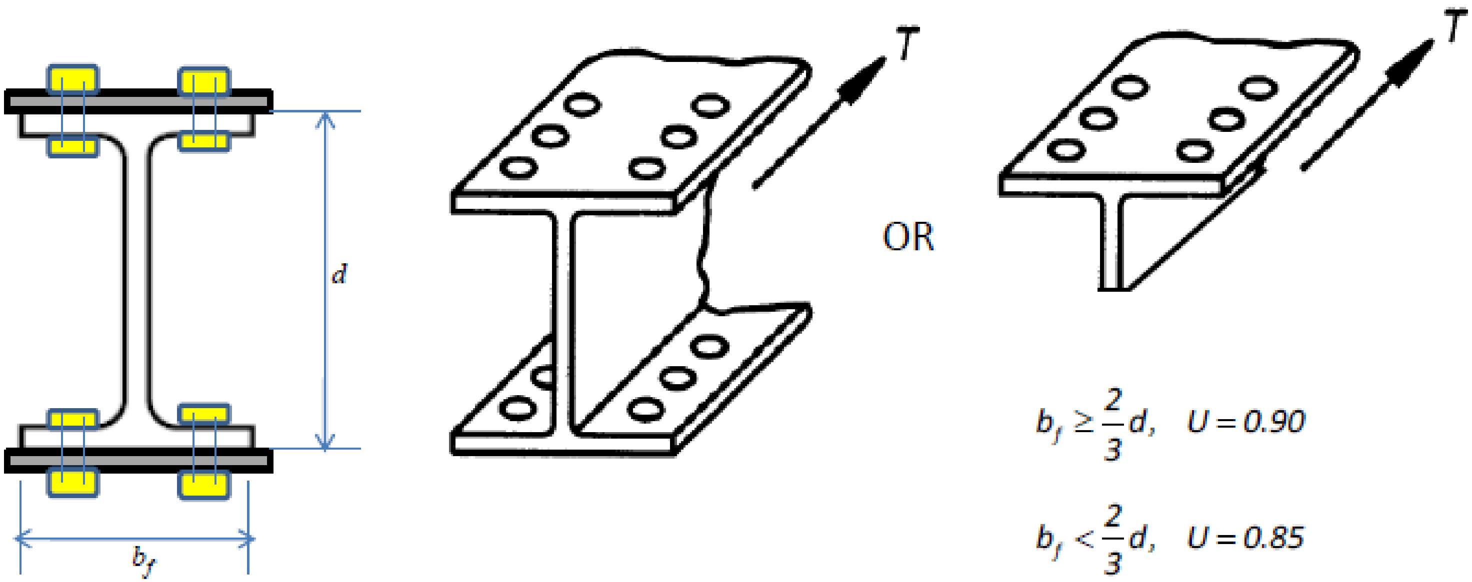


Rectangular HSS – two-sided gusset plate

AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

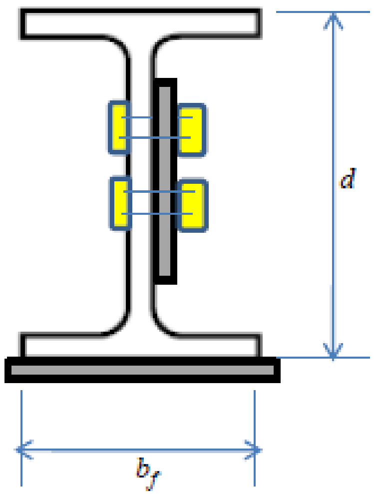


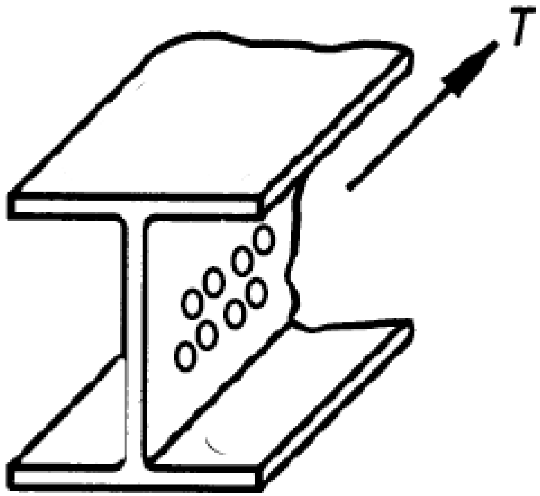
W, M, S or HP, or Tees cut from these shapes-flange connected with three or more fasteners per line in the direction of loading

AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$





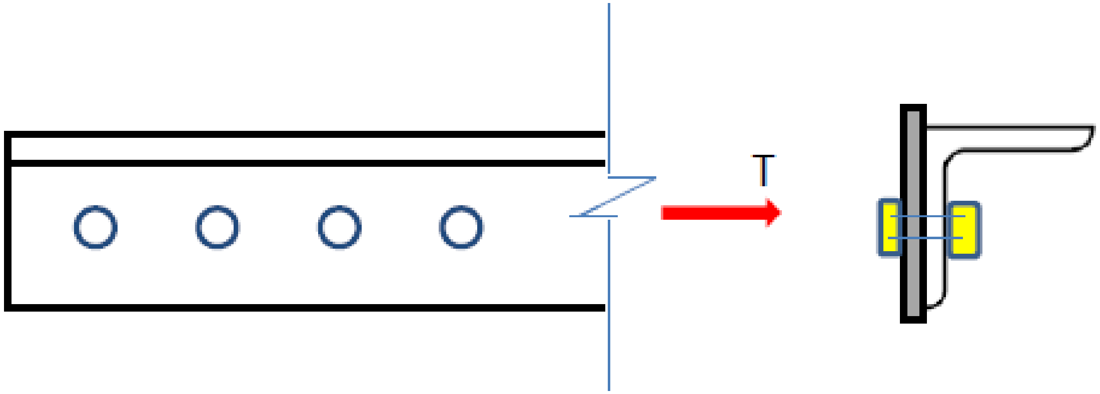
$U = 0.70$

W, M, S or HP, or Tees cut from these shapes-web connected with four or more fasteners per line in the direction of loading

AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$



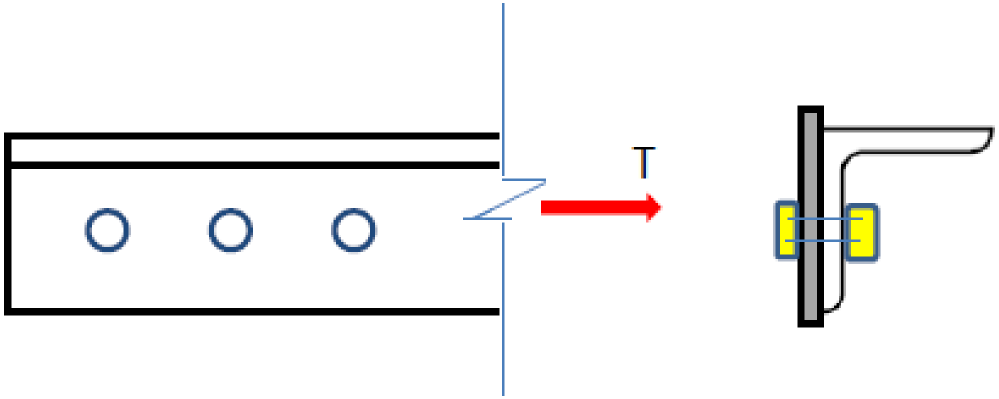
$U = 0.80$

Single angle – four or more fasteners per line in the direction of the load

AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$



$U = 0.60$

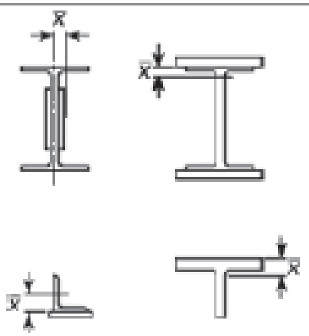
Single angle – three fasteners per line in the direction of the load

AISC 360: C-D3.4

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

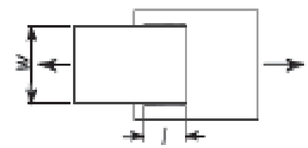
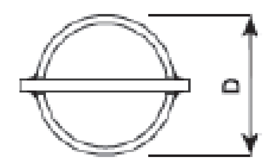
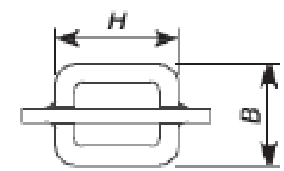
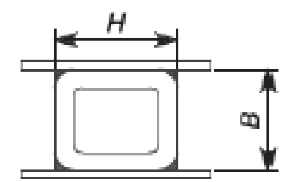
TABLE D3.1
Shear Lag Factors for Connections to Tension Members

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	———

2) Tensile Rupture (Bölüm 7)

Sh

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

Case	Description of Element		Shear Lag Factor, U	Example
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2}{4(B+H)}$	

ANALYSIS OF TENSION MEMBERS

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 the 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 per line in the direction of loading	$U = 0.70$	——
8	Single and double angles (If U is calculated per Case 2, the larger value is permitted to be used.)	with 4 or more fasteners per line in the direction of loading	$U = 0.80$	——
		with 3 fasteners per line in the direction of loading (With fewer than 3 fasteners per line in the direction of loading, use Case 2.)	$U = 0.60$	——

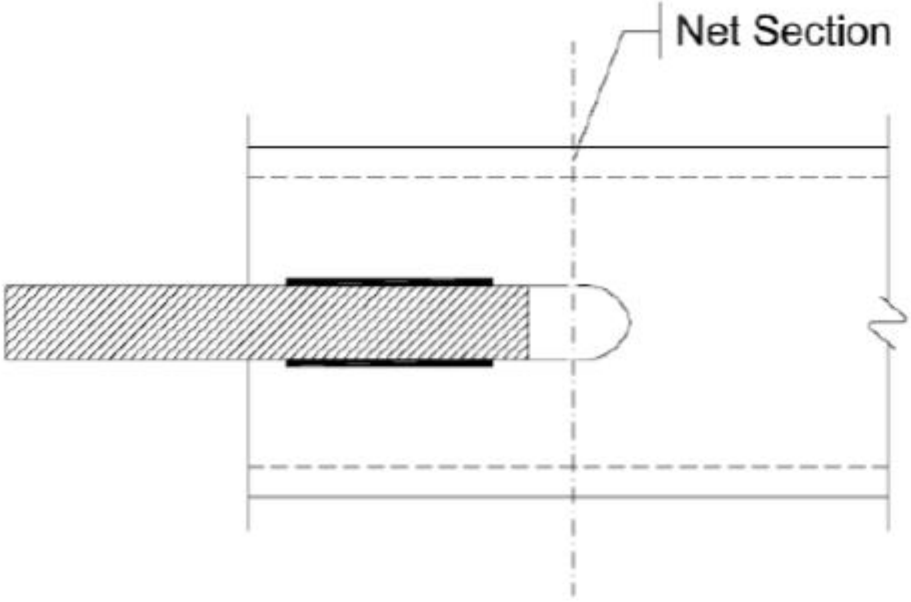
l = length of connection, in. (mm); w = plate width, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm); B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in.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

ANALYSIS OF TENSION MEMBERS

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$

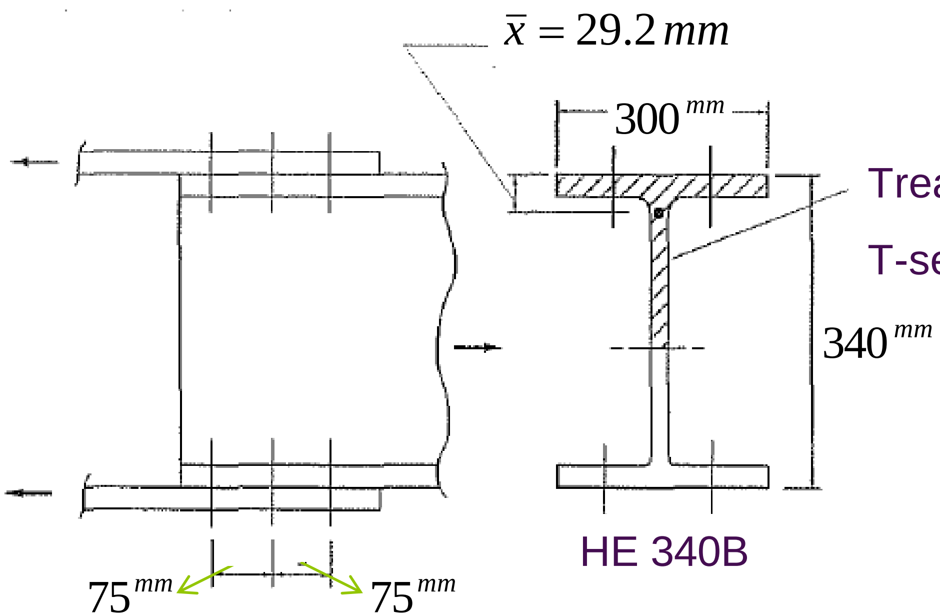
Fig. C-D3.4. Net area through slot for single gusset plate.



For slotted HSS welded to a gusset plate, the net area An, is the gross area minus the product of the thickness and the total width of material that is removed to form the slot.

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$: Example



What is U for computing effective net area for HE340B connected as shown?

$$U = 1 - \frac{29{,}2^{mm}}{150^{mm}} = 0.80$$

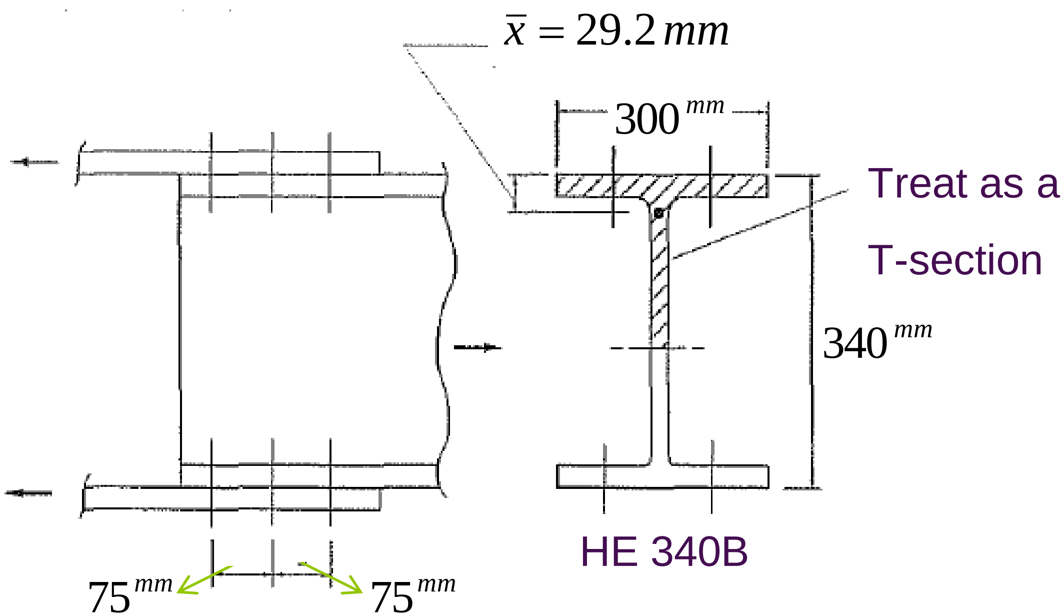
ÇYTYE Table 7.1:

Case 7: I shapes $b_f > ⅔d$ and structural tees cut from these shapes (I/2)

when there are at least three fasteners per line in the direction of stress: U=0.90

2) Tensile Rupture (Bölüm 7)

Shear lag factor, U: $T_n = F_u A_e = F_u U A_n$: Example



What is U for computing effective net area for HE340B connected as shown?

$$U = 1 - \frac{29.2^{mm}}{150^{mm}} = 0.80$$

check for this x-section

$$\frac{b_f}{d} = \frac{\text{flange width}}{\text{section depth}} = \frac{300^{mm}}{340^{mm}} = 0.88 \rangle \frac{2}{3} = 0.667$$ → U=0.90

3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

Failure might occur by shear along a plane through the fastaners plus tension along a perpendicular plane on the area effective in resisting tearing failure. This type of failure is called as “block shear failure”.

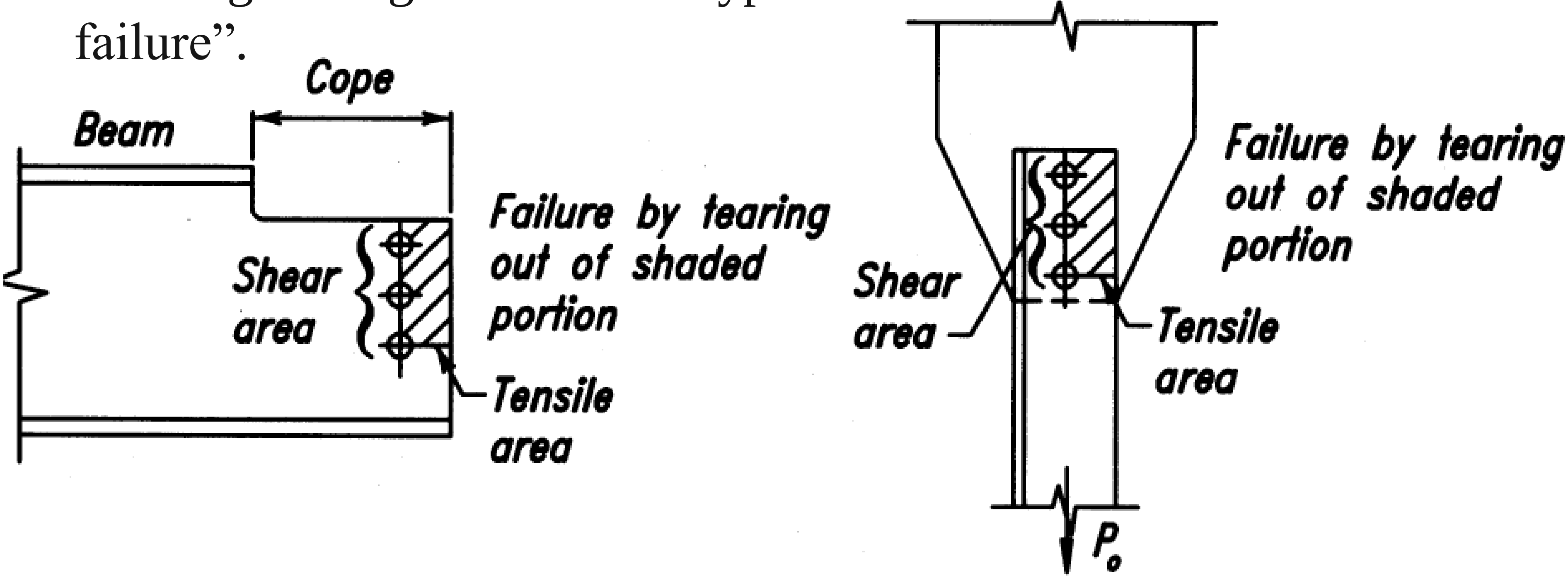
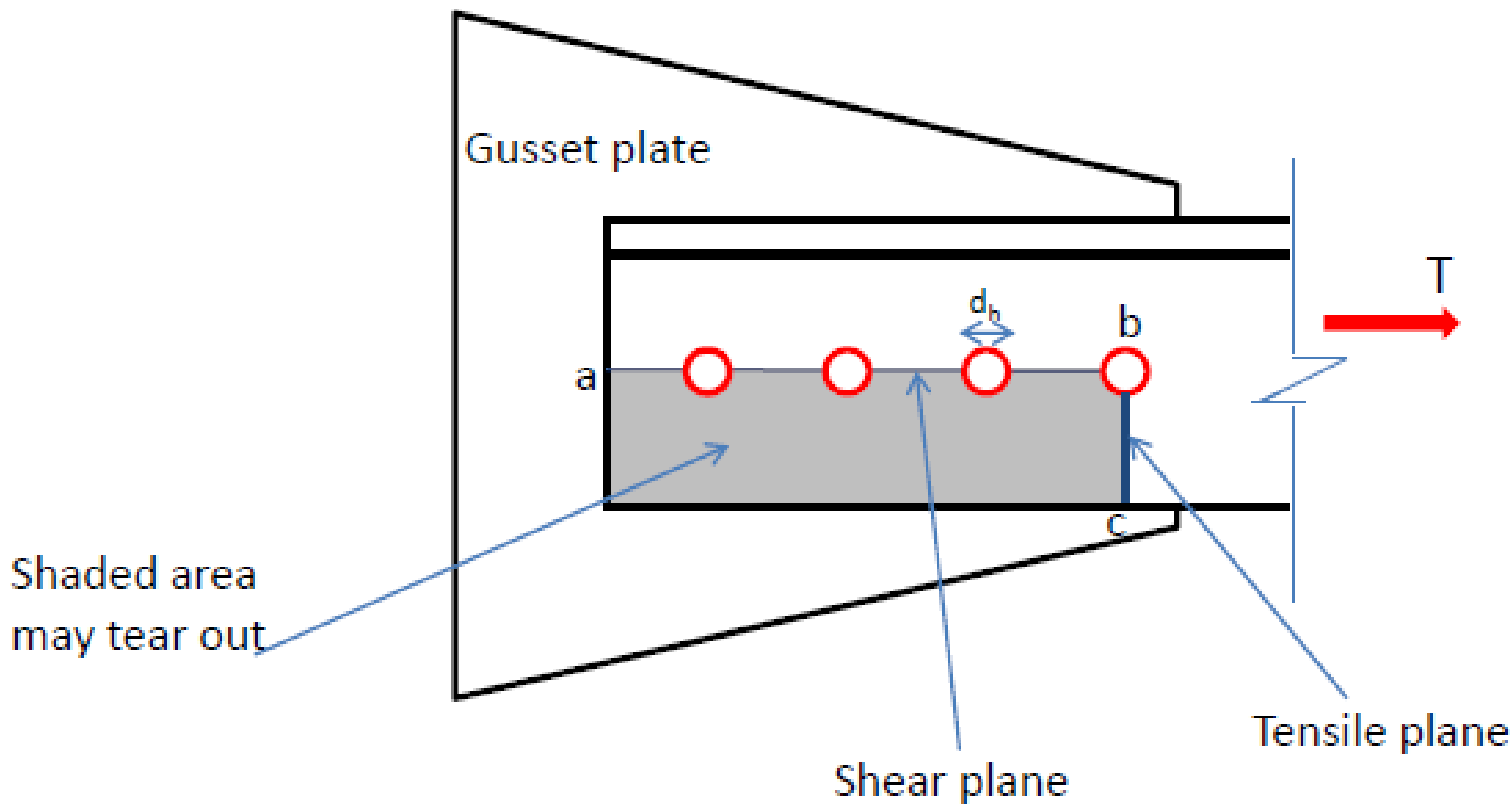


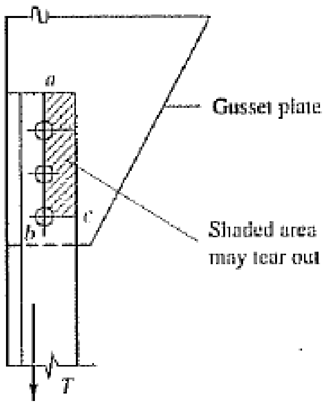
Fig. C-J4.1. Failure surface for block shear rupture limit state.

3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

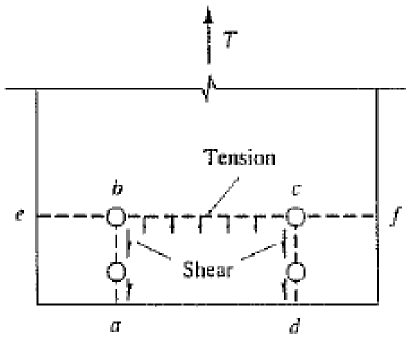


3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

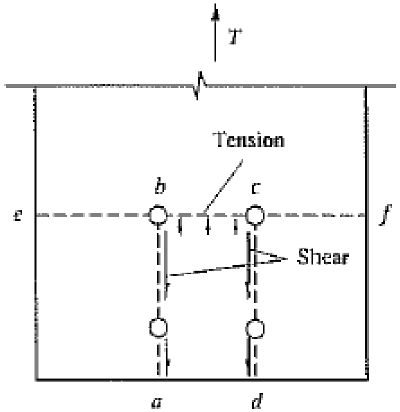
Block shear strength is determined by the summation of the shear and tension terms. Block shear is a rupture or tearing limit state, not a yielding limit state.



(a) Failure by tearing out



(b) Large tension, small shear

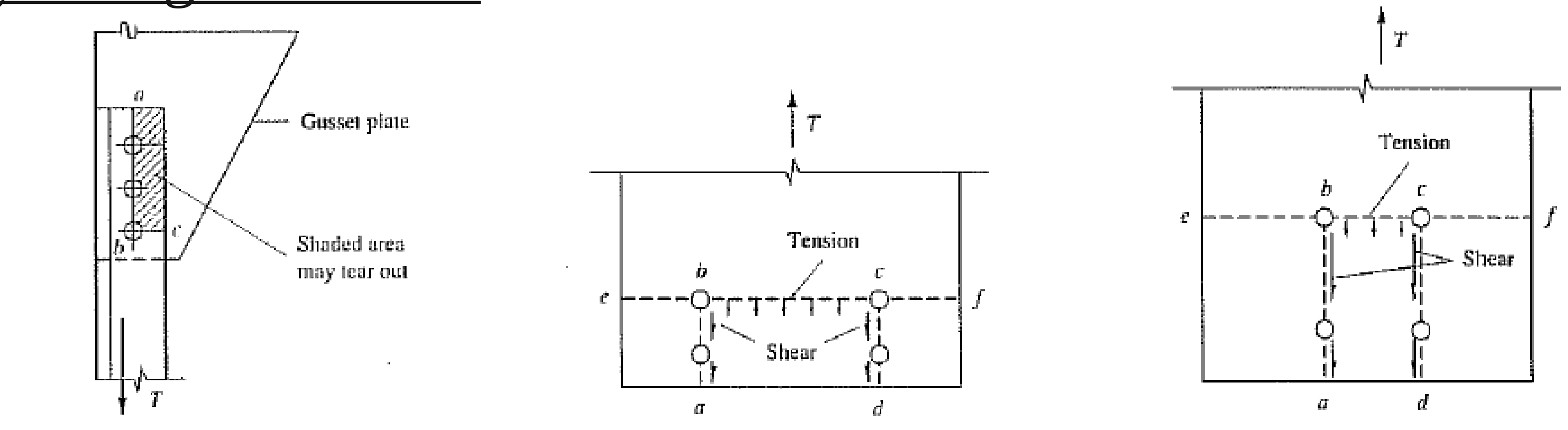


(c) Large shear, small tension

- AISC-J4.3 or ÇYTYE 13.4.3 two block shear failure mode
  - Rupture along the tensile plane accompanied by yielding along the shear planes
  - Rupture along the tensile planes accompanied by rupture along the shear planes

3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

Block shear strength is determined by the summation of the shear and tension terms. Block shear is a rupture or tearing limit state, not a yielding limit state.



(a) Failure by tearing out　(b) Large tension, small shear　(c) Large shear, small tension

- AISC-J4.3 or ÇYTYE 13.4.3 two block shear failure mode
  - Dik çekme yüzeyi boyunca kopma ile kesme yüzeyleri boyunca akma
  - Dik çekme yüzeyi boyunca kopma ile kesme yüzeyleri boyunca kırılma

3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

Nominal Strength T_n is given by:

- Fracture on the net tensile area followed by yielding along the shear plane

$$T_n = 0.60 F_y A_{gv} + U_{bs} F_u A_{nt}$$

- Or, fracture on the net tensile area followed by fracture on the net shear area

$$T_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt}$$

- The smaller strength along the shear planes defines the governing mode of failure

$$T_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \quad (ÇYTYK\ Denk.13.19)$$

When the tension stress is
- uniform U_{bs}=1
- non-uniform U_{bs}=0.5

Non-uniform stress distribution occurs when multiple rows of bolts occur in a beam end connection.

3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

Nominal Strength T_n is given by:

$$T_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \quad (ÇYTYK\,13.19)$$

$$\phi = 0.75 \, (LRFD \; or \; YDKT) \qquad \Omega = 2.0 \, (ASD \; or \; GKT)$$

F_u = specified minimum tensile strength (MPa)
karakteristik çekme gerilmesi

A_{gv} = gross area acted upon by shear (mm^2)
kayma etkisinde kayıpsız enkesit alan

A_{nv} = net area acted upon by shear (mm^2)
kayma etkisinde net (kayıplı) enkesit alanı

A_{nt} = net area acted upon by tension (mm^2)
çekme etkisinde net (kayıplı) enkesit alanı

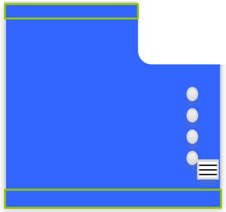
U_{bs} = reduction coefficient
çekme gerilmeleri yayılışını gözönüne alan katsayı

3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)
Block Shear Reduction Factor, U_{bs}

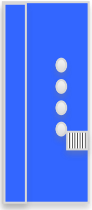
$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$$

- For uniform tensile stress

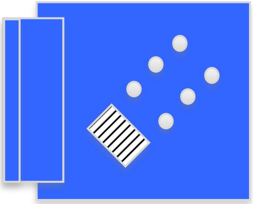
  $U_{bs} = 1.0$



Single-row beam end connections



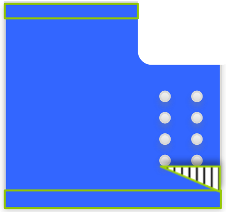
Angle Ends



Gusset Plates

- For non-uniform tensile stress

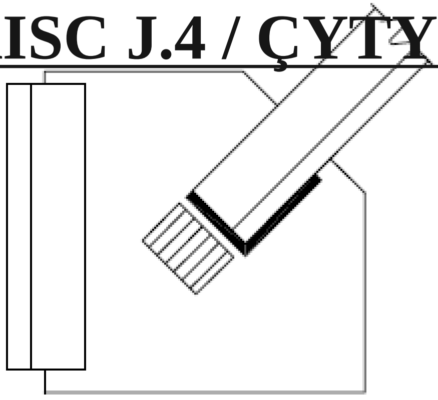
  $U_{bs} = 0.5$



Multiple-row beam end connections

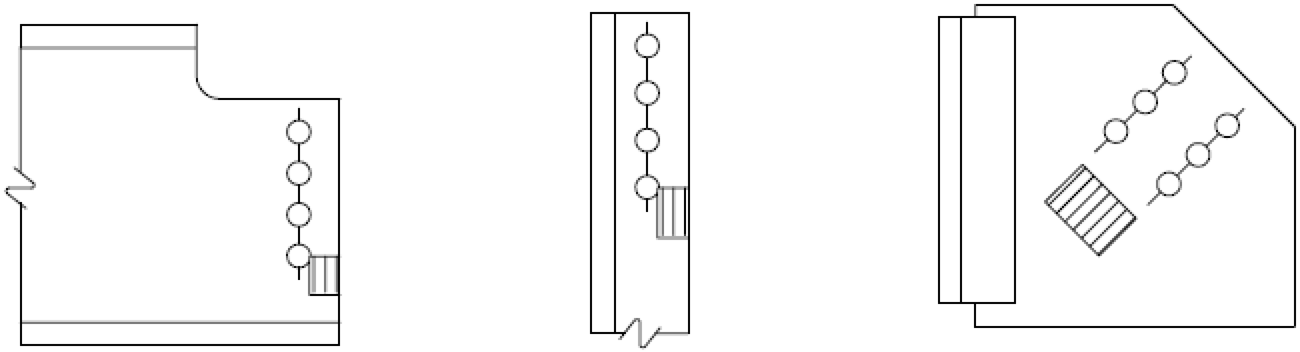
3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

Block Shear Reduction Factor, U_{bs}



Welded Angle

Fig. C-J4.2. Block shear tensile stress distributions.



Single-row beam end connections

Angle Ends

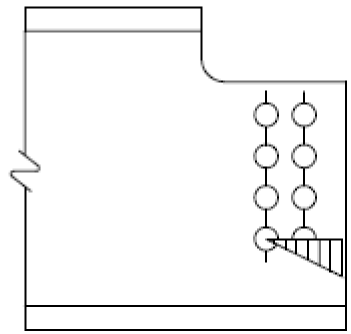
Gusset Plates

(a) Cases for which U_{bs} = 1.0

3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

Block Shear Reduction Factor, U_{bs}

Fig. C-J4.2. Block shear tensile stress distributions.



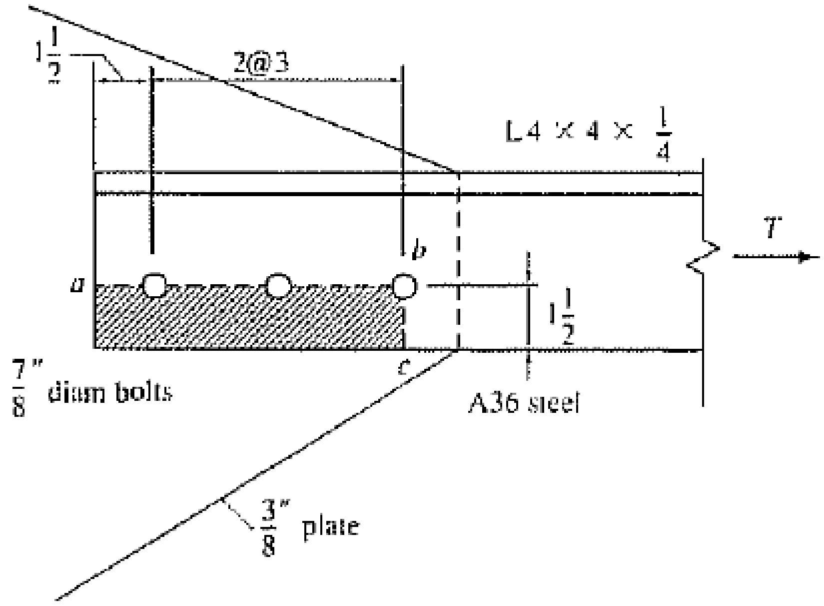
Multiple-row beam end connections

(b) Case for which $U_{bs} = 0.5$

3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

Example

Investigate the block shear failure mode on the angle shown in the connection.



3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

Example

Design strength on general yielding on the gross section:

$$\phi_t T_n = \phi_t F_y A_g = 0.90\left(235^{N/mm2}\right)\left(1550^{mm2}\right) = 327.8 \text{ kN}$$

Design strength on fracture on the net section:

$$\phi_t T_n = \phi_t F_u A_e = \phi_t F_u U A_n$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{27.4}{150} = 0.82$$

$$\phi_t T_n = 0.75\left(360^{N/mm2}\right)(0.82)\left(1550^{mm2} - \left[20^{mm} + 2^{mm} + 2^{mm}\right] x 8^{mm}\right)$$

$$= 300.6 \text{ kN}$$

3) Block Shear Failure (AISC J.4 / ÇYTYE 13.4.3)

Example

Design strength due to block shear limit state:

$$A_{nv} = (\text{length a - b less 2.5 holes}) \times \text{thickness}$$
$$= \left[187.5^{\text{mm}} - 2.5\left(20^{mm} + 4^{mm}\right)\right] x8^{mm} = 1020\ \text{mm}^2$$

$$A_{nt} = (\text{length b - c less 0.5 holes}) \times \text{thickness}$$
$$= \left[37.5^{\text{mm}} - 0.5\left(20^{mm} + 4^{mm}\right)\right] x8^{mm} = 204\ \text{mm}^2$$

$$A_{gv} = (\text{length a - b}) \times \text{thickness}$$
$$= \left[187.5^{\text{mm}}\right] x8^{mm} = 1500\ \text{mm}^2$$

Compare

$$\begin{bmatrix} 0.6F_y A_{gv} = 0.6\left(235^{N/mm2}\right)1500^{mm2} \\ = 211.5\,kN \end{bmatrix} < \begin{bmatrix} 0.6F_u A_{nv} = 0.6\left(360^{N/mm2}\right)1020^{mm2} \\ = 220.3\,kN \end{bmatrix}$$

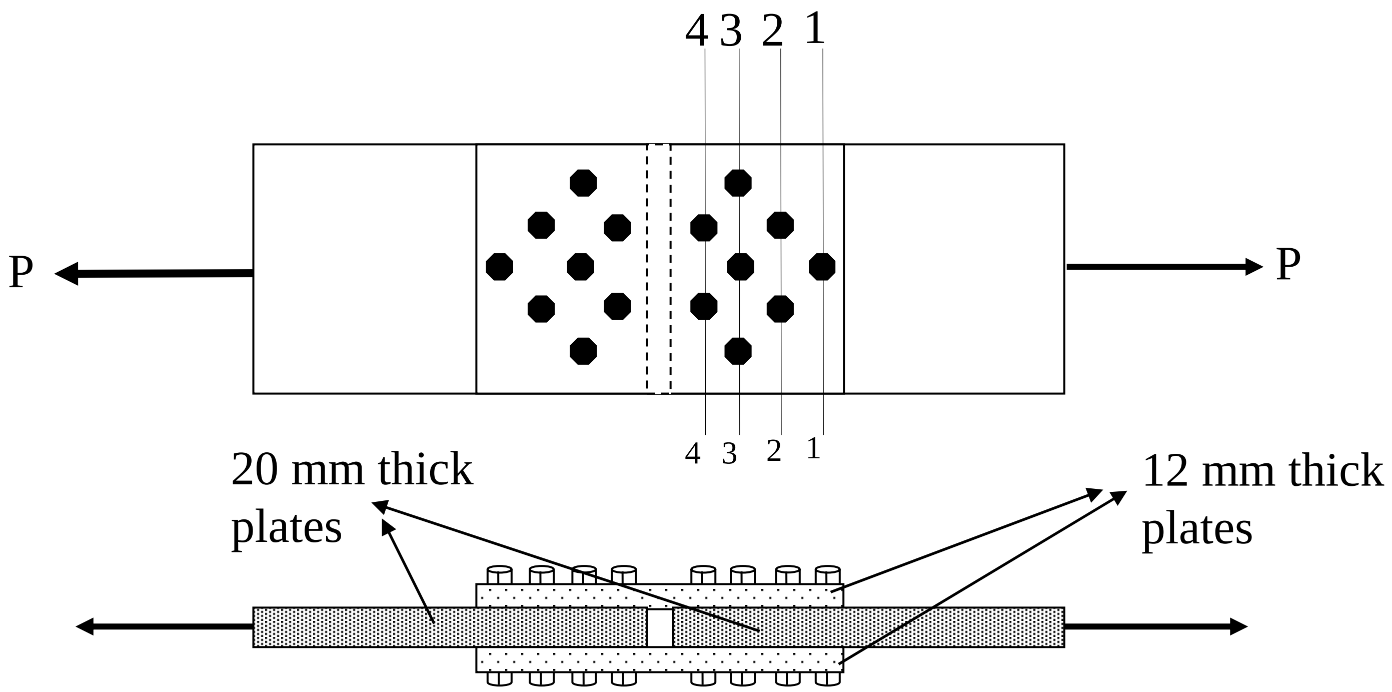
ÇYTYE 13.9

$$T_n = 0.6F_y A_{gv} + F_u U_{bs} A_{nt}$$
$$= 211500^{N} + 360^{N/mm2}(1.0)\left(37.5^{mm} - 0{,}5x24^{mm}\right)x8^{mm}$$
$$= 284.9\ \text{kN}$$

$$\phi T_n = 0.75(284{,}9) = \boxed{213.7\ \text{kN}}$$

Load Transfer at Connections

Each equal size fastener transfers an equal share of load if fasteners are symmetric with respect to the centroidal axis of tension member.

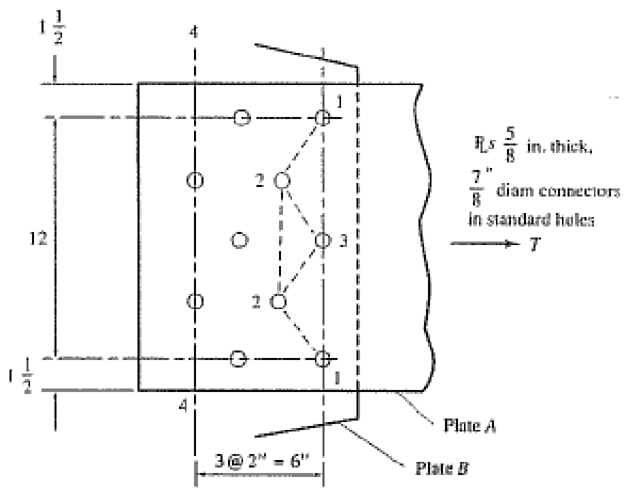


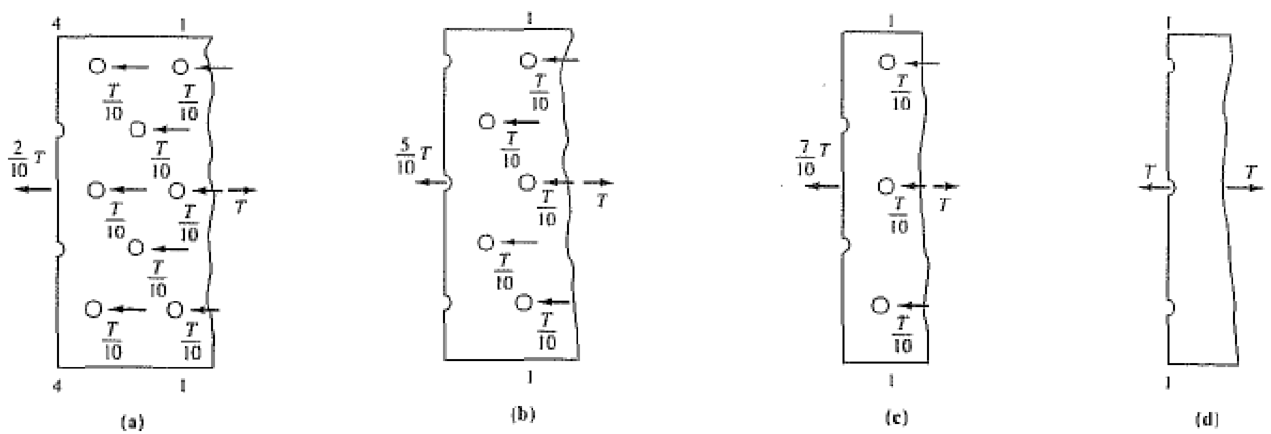
Load Transfer at Connections

Example: Calculate the required strength (LRFD), T_u, for plate A of the single lap

S235 Steel

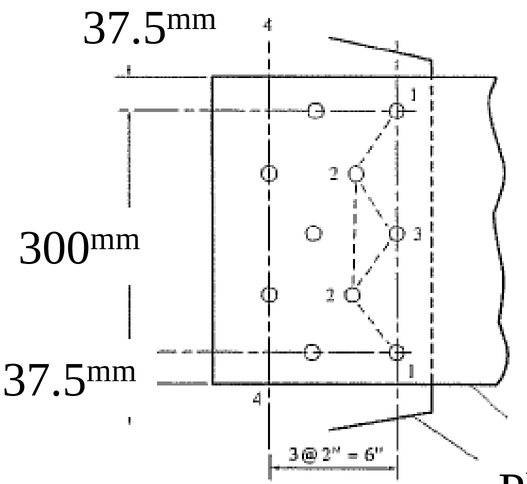
F_u = 360 MPa





Load Transfer at Connections

Example: Net area calculations



PLs thickness = 15^{mm}
M22 bolts in standard holes

Plate A
Plate B

Deduction in width for 1 hole:

$$= \phi_{hole} + 2^{mm} = \phi_{bolt} + 4^{mm} = 26\,mm$$

$$A_{n(1-1)} = 15^{mm}\left(375^{mm} - 3x26^{mm}\right)$$

$$= 4445\,mm^2\,(Under\,1.0T)$$

$$A_{n(1-2-3-2-1)} = 15^{mm}\left[375^{mm} - 5\left(26^{mm}\right) + 4\frac{\left(50^{mm}\right)^2}{4\left(75^{mm}\right)}\right]$$

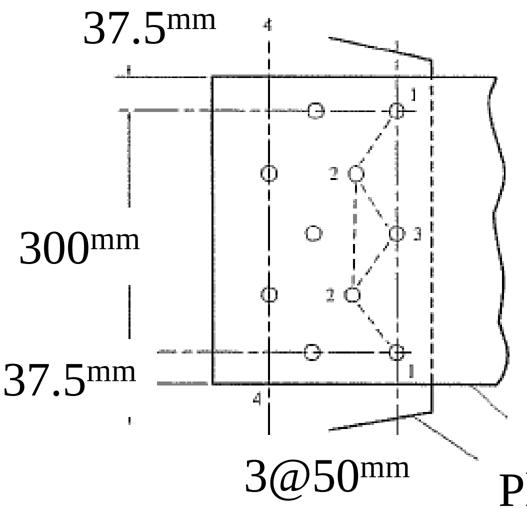
$$= 4175\,mm^2\,(Under\,1.0T)$$

$$A_{n(1-2-2-1)} = 15^{mm}\left[375^{mm} - 4\left(26^{mm}\right) + 2\frac{\left(50^{mm}\right)^2}{4\left(75^{mm}\right)}\right]$$

$$= 4315\,mm^2\,(Under\,0{,}9T)$$

Load Transfer at Connections

Example: Required strength on fracture on the net area



PLs thickness = 15mm
M22 bolts in standard holes

$$T_{n(1-2-3-2-1)} = 360^{N/mm2} x4175^{mm^2}$$

$$T_{n(1-2-3-2-1)} = 1523.8\,kN$$

$$T_{u(1-2-3-2-1)} = \phi T_n = 0.75x1523.8kN$$

$$T_{u(1-2-3-2-1)} = 1142.9\,kN$$

$$T_{n(1-2-2-1)} = 360^{N/mm2} x4315^{mm^2}$$

$$T_{n(1-2-2-1)} = 1553\,kN$$

$$0.9T_{u(1-2-2-1)} = \phi T_n = 0.75x1553kN$$

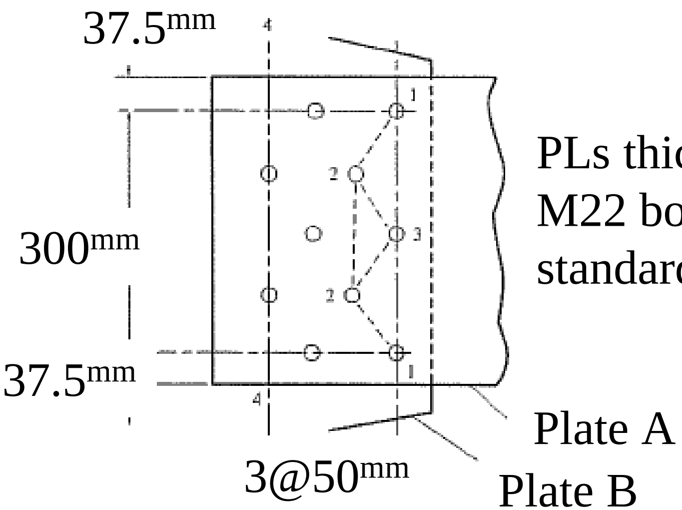
$$0.9T_{u(1-2-2-1)} = 1165.0\,kN$$

$$T_{u(1-2-2-1)} = 1294.5\,kN$$

ANALYSIS OF TENSION MEMBERS

Load Transfer at Connections

Example: Required strength on general yielding on the gross area



PLs thickness = 15mm
M22 bolts in
standard holes

$$T_u = \phi T_n = 0.90 F_y A_g$$

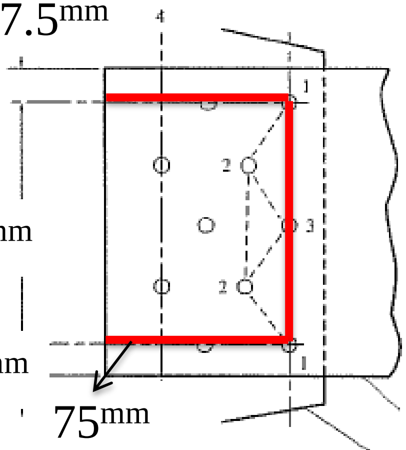
$$T_n = F_y A_g = \left(235^{N/mm2}\right)\left(375^{mm} x 15^{mm}\right)$$

$$T_n = 1321.9 \text{ kN}$$

$$T_u = \phi T_n = 0.9 x 1321.9 = 1189.6 kN$$

Load Transfer at Connections

Example: Required strength on general yielding on the gross area



PLs thickness = 15mm
M22 bolts in standard holes

$$A_{nv} = 2x\left[75^{mm} + 150^{mm} - 1.5\left(22^{mm} + 4^{mm}\right)\right]x15^{mm}$$

$$A_{nv} = 5580\ mm^2$$

$$A_{gv} = 2x\left[75^{mm} + 150^{mm}\right]x15^{mm} = 6750\ mm^2$$

$$A_{nt} = \left[300^{mm} - 2\left(22^{mm} + 4^{mm}\right)\right]x15^{mm} = 3720\ mm^2$$

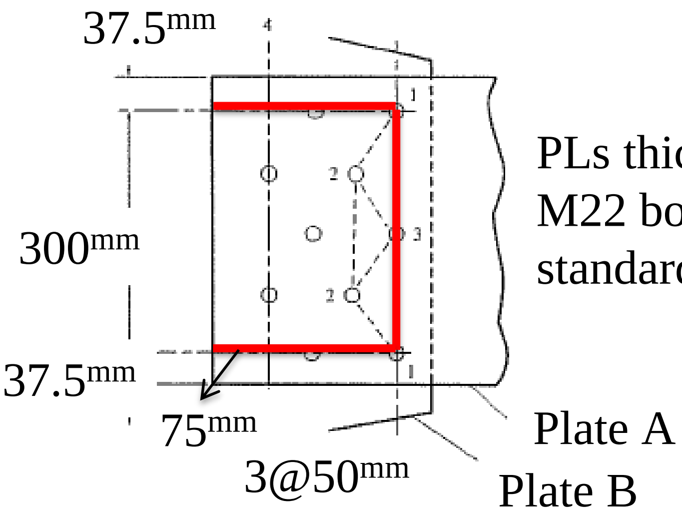
$$\begin{bmatrix} 0.6F_y A_{gv} = 0.6\left(235^{N/mm2}\right)6750^{mm2} \\ = 951{,}8\,kN \end{bmatrix} < \begin{bmatrix} 0.6F_u A_{nv} = 0.6\left(360^{N/mm2}\right)5580^{mm2} \\ = 1205{,}3\,kN \end{bmatrix}$$

$$T_n = \left(0.6F_y A_{gv} + F_u U_{bs} A_{nt}\right) = 951800^{N} + 360^{N/mm^2} x1.0x3720^{mm^2} = 2291.6\ kN$$

$$T_u = \phi T_n = 0.75x2291.6^{kN} = 1718.7kN$$

Load Transfer at Connections

Example: Summary



PLs thickness = 15mm
M22 bolts in standard holes

Summary:

Tensile Yielding: $T_u = 1189.6\,kN$

Tensile Rupture: $T_{u(1-2-3-2-1)} = \boxed{1142.9\,kN}$

Block Shear (f): $T_u = 1718.3\,kN$

Summary and Example

$$P_u \leq \phi_t P_n \quad (LRFD)$$

$$P_a \leq \frac{P_n}{\Omega} \quad (ASD)$$

Summary and Example

The design tensile strength $\phi_t P_n$ or allowable tensile strength P_n / Ω will be the lower value obtained according to the the limit states of (ANSI/AISC 360-10, D2):

a) Tensile yielding in gross area:

$$\phi_t P_n = \phi_t F_y A_g = 0.90 F_y A_g \quad (LRFD)$$

$$\frac{P_n}{\Omega} = \frac{F_y A_g}{\Omega} = \frac{F_y A_g}{1.67} \quad (ASD)$$

b) Tensile rupture in the net area:

$$\phi_t P_n = \phi_t F_u A_e = 0.75 F_u A_e \quad (LRFD)$$

$$\frac{P_n}{\Omega} = \frac{F_u A_e}{\Omega} = \frac{F_u A_e}{2.0} \quad (ASD)$$

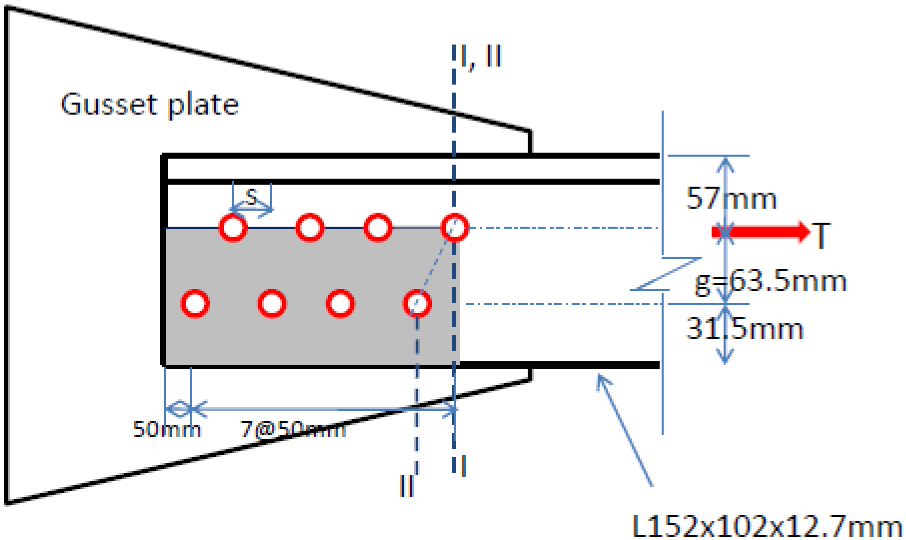
Summary and Example

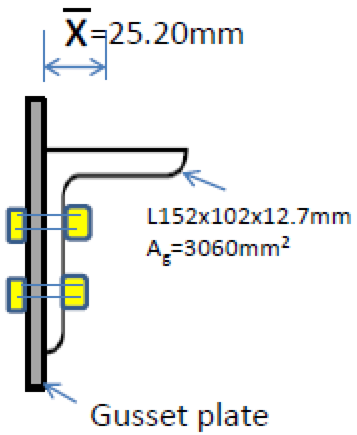
c) Block shear design strength (AISC 360-10, J4.3):

$$\phi R_n = 0.75(0.6F_u A_{nv} + U_{BS} F_u A_{nt}) \leq 0.75(0.6F_y A_{gv} + U_{BS} F_u A_{nt}) \quad (LRFD)$$

$$\frac{R_n}{\Omega} = \left\{ \frac{0.6F_u A_{nv} + U_{BS} F_u A_{nt}}{2.0} \right\} \leq \left\{ \frac{0.6F_y A_{gv} + U_{BS} F_u A_{nt}}{2.0} \right\} \qquad (ASD)$$

Summary and Example





Given: Steel Grade 50 (Fe52) ➔ F_y=345 Mpa, F_u=485 Mpa
d_h=22mm in standart hole
Live Load / Dead Load = 3.0

Required: Service Load (Dead Load + Live Load)

Summary and Example

1) Areas for different limit states:

$$s = 50mm \quad g = 63.50mm \quad l = 350mm \quad L = 400mm(7 @ 50 + 50)$$

$$Section\ I - I : A_n = A_g - n(d_{hd}t) = 3060 - 1(22+2)12.7 = 2755.2mm^2$$

$$Section\ II - II : A_n = A_g - n(d_{hd}t) + \frac{s^2}{4g}t$$

$$= 3060 - 2(22+2)12.7 + \frac{50^2}{4(63.50)}12.7 = 2325.4mm^2$$

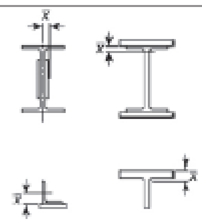
$$A_n = 2325.4mm^2 \quad (smaller)$$

Summary and Example

Reduction for eccentricity-caused non-uniform stress (shear lag):

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{25.20}{350} = 0.93 > 0.90 \Rightarrow use \quad U = 0.9$$

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

Case	Description of Element		Shear Lag Factor, U	Example
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.)		$U = 1 - \bar{x}/l$	
8	Single and double angles (If U is calculated per Case 2, the larger value is permitted to be used.)	with 4 or more fasteners per line in the direction of loading	$U = 0.80$	——
		with 3 fasteners per line in the direction of loading (With fewer than 3 fasteners per line in the direction of loading, use Case 2.)	$U = 0.60$	——

If we used case 8 for this problem, it'd be conservative

Summary and Example

Reduction for eccentricity-caused non-uniform stress (shear lag):

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{25.20}{350} = 0.93 > 0.90 \Rightarrow use \quad U = 0.9$$

$$A_e = U.A_n = 0.9(2325.4) = 2093\,mm^2$$ Effective net area

$$A_{gv} = 400(12.70) = 5080\,mm^2$$ gross area subjected to shear

$$A_{nv} = [400 - 3.5(22+2)](12.70) = 4013\,mm^2$$ net area subjected to shear

$$A_{nt} = \left[(63.50 + 31.50) - \frac{1}{2}(22+2)\right](12.70) = 1054\,mm^2$$ net area subjected to tension

Summary and Example

2) Design Strength:

a. Yielding Limit State

$$\phi_t P_n = \phi_t F_y A_g = 0.9(345)(3060) = 950kN \quad (LRFD)$$

$$\frac{P_n}{\Omega} = \frac{F_y A_g}{\Omega} = \frac{(345)(3060)}{1.67} = 632kN \quad (ASD)$$

Summary and Example

b. Rupture Limit State

$$\phi_t P_n = \phi_t F_u A_e = 0.75(485)(2093) = 761\,kN \quad (LRFD)$$

$$\frac{P_n}{\Omega} = \frac{F_u A_e}{\Omega} = \frac{(485)(2093)}{2.00} = 508\,kN \quad (ASD)$$

Summary and Example

c. Block Shear Limit State

$$\phi R_n = 0.75(0.6F_u A_{nv} + U_{BS} F_u A_{nt}) \le 0.75(0.6F_y A_{gv} + U_{BS} F_u A_{nt}) \quad (LRFD)$$

$$= 0.75\{0.6(485)(4013) + 1.0(485)(1054)\} \le 0.75\{0.6(345)(5080) + 1.0(485)(1054)\}$$

$$= 1259kN \le 1172kN \Rightarrow \phi R_n = 1172kN \quad (LRFD)$$

$$\frac{R_n}{\Omega} = \left\{\frac{0.6F_u A_{nv} + U_{BS} F_u A_{nt}}{2.0}\right\} \le \left\{\frac{0.6F_y A_{gv} + U_{BS} F_u A_{nt}}{2.0}\right\} \quad (ASD)$$

$$\frac{R_n}{\Omega} = \left\{\frac{0.6(485)(4013) + 1.0(485)(1054)}{2.0}\right\} \le \left\{\frac{0.6(345)(5080) + 1.0(485)(1054)}{2.0}\right\}$$

$$= 839.5kN \le 782kN \Rightarrow \frac{R_n}{\Omega} = 782kN \quad (ASD)$$

Summary and Example

Final design strength ➔

$$\phi_t P_n = \min(950kN, 761kN, 1172kN) = 761kN \quad (LRFD)$$

$$\frac{P_n}{\Omega} = \min(632kN, 508kN, 782kN) = 508kN \quad (ASD)$$

Summary and Example

3) Demand (P_u & P_a):

$$LRFD \Rightarrow P_u = 1.4DL$$

$$P_u = 1.2DL + 1.6LL + 0.5(L_r \text{ or } S \text{ or } R)$$

$$= 1.2DL + 1.6(3DL) = 6DL$$

6DL controls

$$let \quad \phi_t P_n = P_u$$

$$761 = 6DL \Rightarrow DL = 126.8kN$$

$$LL = 3DL = 380.5kN$$

$$Total \quad Service \quad Load \Rightarrow LL + DL = 507.3kN$$

Summary and Example

$$ASD \Rightarrow P_a = DL + LL$$

$$let \quad \frac{P_n}{\Omega} = P_a$$

$$508 = DL + 3DL = 4DL \Rightarrow DL = 127\,kN$$

$$Total \quad Service \quad Load \Rightarrow LL + DL = 508\,kN$$

DESIGN OF TENSION MEMBERS

Considerations

‘The design of tension members will require consideration of failure modes not specifically addressed here.

For tension members with welded connections, the design strength of the welds in shear and tension must be considered.

For tension members with bolted connections, the design strength of the bolts in shear, tension, and bearing must be considered.

Load eccentricity effects at the connection points also must be considered.’

DESIGN OF TENSION MEMBERS

<u>Considerations</u>

'In this Chapter, the design strength in tension of the actual member only is considered. The reader is referred to Chapters 9 and 10 for the design of the connections for tension members.

Tension members need to have enough gross cross-sectional area for strength in yielding and enough effective area for strength in fracture. Note that the effective area accounts for shear lag effects.

In addition to having enough design strength in yielding and fracture, block shear at the connected ends needs to be checked.

In some cases, there might be more than one mode of failure in block shear. '

DESIGN OF TENSION MEMBERS

Considerations

‘The design strength of the tension member is the smallest of the strength in yielding, fracture, and block shear.

Slenderness effects should also be considered. The AISC specification recommends a slenderness limit *L*/*i* of 300 to prevent flapping, flutter, or sag of the member, but this is not a mandatory requirement. If this slenderness limit cannot be met, the member can be pretensioned to reduce the amount of sag.’

DESIGN OF TENSION MEMBERS

The design of a tension member can be summarized as follows:

1. Determine the minimum gross area from the tensile yielding failure mode equation:

$$A_g \geq \frac{T_u}{0.9F_y}$$

2. Determine the minimum net area from the tensile fracture failure mode equation:

$$A_n \geq \frac{T_u}{0.75F_uU}$$

$$A_n = A_g - \Sigma A_{holes} \geq \frac{T_u}{0.75F_uU}$$

where the net area can be found by:

$$A_g \geq \frac{P_u}{0.75F_uU} + \Sigma A_{holes}$$

(Abi Aghayere and Jason Vigil (2015))

DESIGN OF TENSION MEMBERS

The design of a tension member can be summarized as follows:

3. Use the larger A_g value and select a trial member size based on the larger value of A_g.

4. For tension members, AISC specification Section D1 suggests that the slenderness ratio KL/i_{min} should be $\leq$ 300 to prevent flapping or flutter of the member,

where:

K = effective length factor (usually assumed to be 1.0 for tension members),

L = unbraced length of the tension member

i_{min} = smallest radius of gyration of the member.

(Abi Aghayere and Jason Vigil (2015))

DESIGN OF TENSION MEMBERS

<u>The design of a tension member can be summarized as follows:</u>

The smallest radius of gyration for rolled sections can be obtained the Archelor Catalog. For other sections, such as plates, the radius of gyration can be calculated from:

$$i_{\min} = \sqrt{\frac{I_{\min}}{A_g}} \rangle \frac{L}{300}$$

where I_{min} is the smallest moment of inertia.

If the above equation cannot be satisfied (i.e., the member is too slender), the member should be pretensioned. Allow for 5% to 10% pretension force in the design of the member.

(Abi Aghayere and Jason Vigil (2015))

DESIGN OF TENSION MEMBERS

The design of a tension member can be summarized as follows:

5. Determine the block shear capacity of the selected tension member.

If ϕP_n (block shear) is greater than P_u, the member is adequate.
If ϕP_n (block shear) is less than P_u, increase the member size and repeat step 5 until ϕP_n (block shear) is $\geq P_u$

(Abi Aghayere and Jason Vigil (2015))

DESIGN OF TENSION MEMBERS

Example 1:

Design the X-brace in the first story of the building which is subjected to wind loads. Use a steel plate that conforms to EN10025-S275.

11.4 kips = 50.7 kN

18.2 kips = 81.0 kN

17.9 kips = 79.7 kN

28.6 kips = 127.3 kN

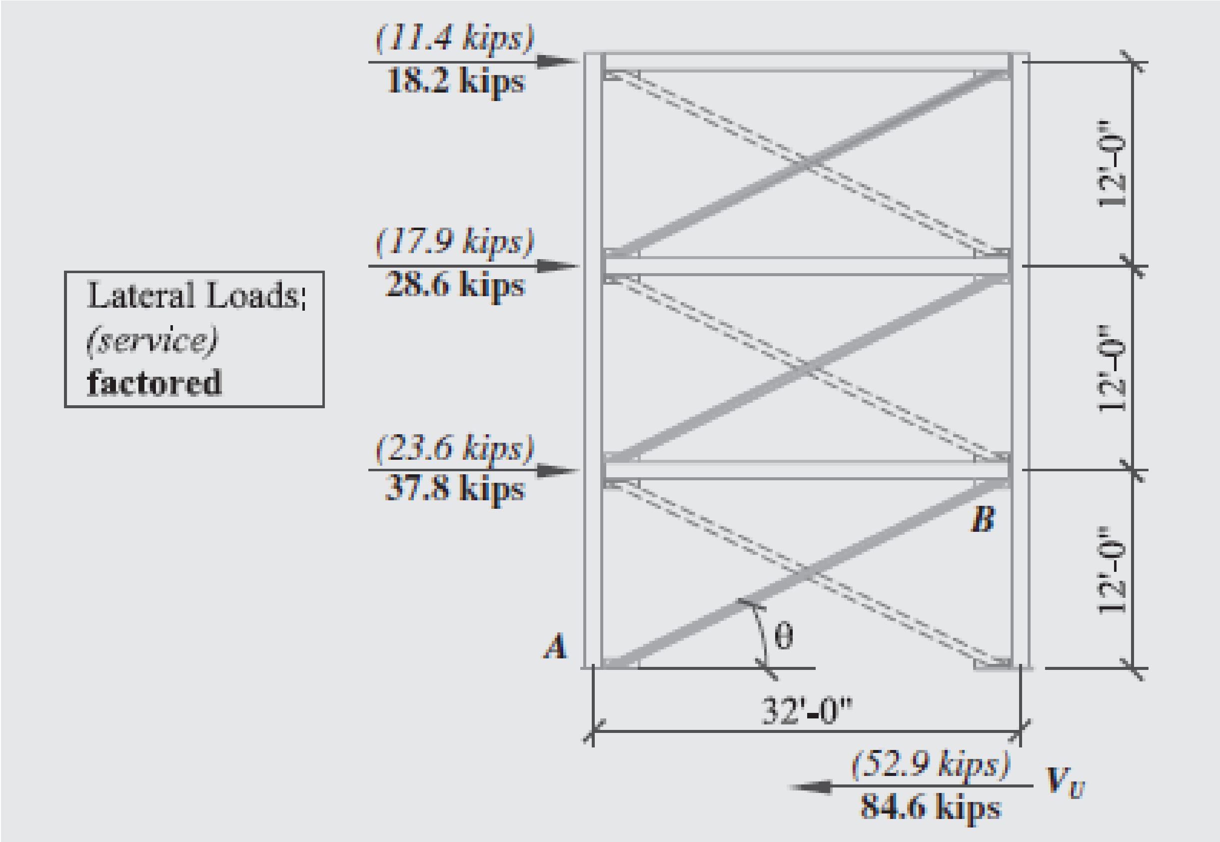
23.6 kips = 105.0 kN

37.8 kips = 168.2 kN

52.9 kips = 235.4 kN

32 ft = 9.75 m

12 ft = 3.65 m



DESIGN OF TENSION MEMBERS

Example 1:

Some X-brace configurations have slender members such that they can only support loads in tension. In this problem, all of the members are assumed to be too slender to support compression loads. Only the shaded members support lateral loads in the assigned direction of the lateral loads.

The lateral loads shown below are the loads acting on each X-braced frame. The wind loads acting on the entire building must be distributed to the various braced frames in the building in the direction of the lateral load. If the diaphragm is assumed to be rigid, the lateral load is distributed in proportion to the stiffness of each braced frame. If the diaphragm is flexible, the lateral load is distributed in proportion to the tributary area of each braced frame.

Example 1: Solution

Maximum load factor for wind is 1.6 (ÇYTYE 2016)

Loads at Each Level:

Service Loads	Factored Loads
P_r = 50.7 kN	(1.6) 50.7 kN = 81.0 kN
P_3 = 79.7 kN	(1.6) 79.7 kN = 127.3 kN
P_2 = 105.0 kN	(1.6) 105.0 kN = 168.2 kN

V_u (base shear) = $81.0^{kN} + 127.3^{kN} + 168.2^{kN} = 376.5$ kN

$\theta = \tan^{-1}(3.65^{m}/9.75^{m}) = 20.6^{\circ}$

$T_{AB} = V_u / \cos\theta = 376.5^{kN} / \cos(20.6^{\circ}) = 402.2$ kN

Example 1: Solution

1- $A_g \geq \frac{(P_u)}{0.9F_y}$ (allow for 5% to 10% pretension: use 7.5%pretension)

$$A_g \geq \frac{\left(402200^{N}\right)(1.075)}{0.9x275^{N/mm^2}} = 1747mm^2$$

Try a $13^{mm} \times 150^{mm}$ plate, $A_g = 1950\ mm^2$

Example 1: Solution

2- Number of bolts: Bolts and welds will be covered in Section 8, but for now, assume the shear strength of 8.8 bolts in single shear with threads not excluded from shear planes to be as follows:

ϕR_n = 54.3 kN for M16 bolts

ϕR_n = 84.8 kN for M20 bolts

5 M16 bolts: $\phi R_n = (5)(54.3^{kN}) = 271.5 \text{ kN} \leq 376.5 \text{ kN}$

6 M16 bolts: $\phi R_n = (6)(54.3^{kN}) = 325.8 \text{ kN} \leq 376.5 \text{ kN}$

5 M20 bolts: $\phi R_n = (5)(84.8^{kN}) = 424 \text{ kN} \geq 376.5 \text{ kN}$

Use five M20 bolts in a single line with threads not excluded from shear planes.

The shear lag factor, U, is 1.0 for plates connected with bolts (from Şekil 13.9, ÇYTYE 2016).

Example 1: Solution

2- $A_g \geq \dfrac{(P_u)}{0.75 F_u U} + \Sigma A_{holes}$

$$A_g \geq \frac{\left(376500^{N}\right)(1.075)}{0.75x430^{N/mm^2}\,x1.0} + (1\,hole)\left(20^{mm} + 2^{mm} + 2^{mm}\right)\left(13^{mm}\right) = 1567\,mm^2$$

3- From step 1, A_g = 1635 mm^2 and from step 2, A_g = 1567 mm^2; both are less than the gross area of the trial member size for 1950 mm^2.

Example 1: Solution

4- Check slenderness ratio:

$$L=\sqrt{\left(9.75^{m}\right)^{2}+\left(3.65^{m}\right)^{2}}=10.4m$$

$$I_{\min}=\frac{bh^{3}}{12}=\frac{\left(150^{mm}\right)\left(13^{mm}\right)^{3}}{12}=27463mm^{4}$$

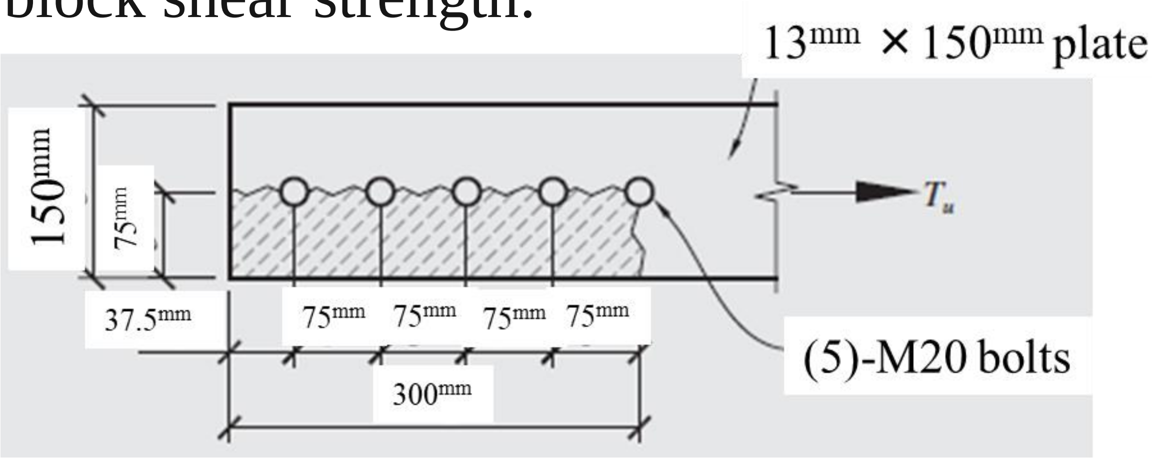
$$i_{\min}=\sqrt{\frac{I_{\min}}{A}}\leq ?\frac{L}{300}$$

$$i_{\min}=\sqrt{\frac{27463^{mm^{4}}}{1950^{mm^{2}}}}=3.75^{mm}\rangle\frac{10400^{mm}}{300}=34.7$$

Slenderness limit is exceeded. The assumption to pretension the X-brace is justfied.

Example 1: Solution

4- Check block shear strength:



$$A_{gv} = (300^{mm})(13^{mm}) = 3900mm^2$$

$$A_{nv} = A_{gv} - \Sigma A_{holes}$$

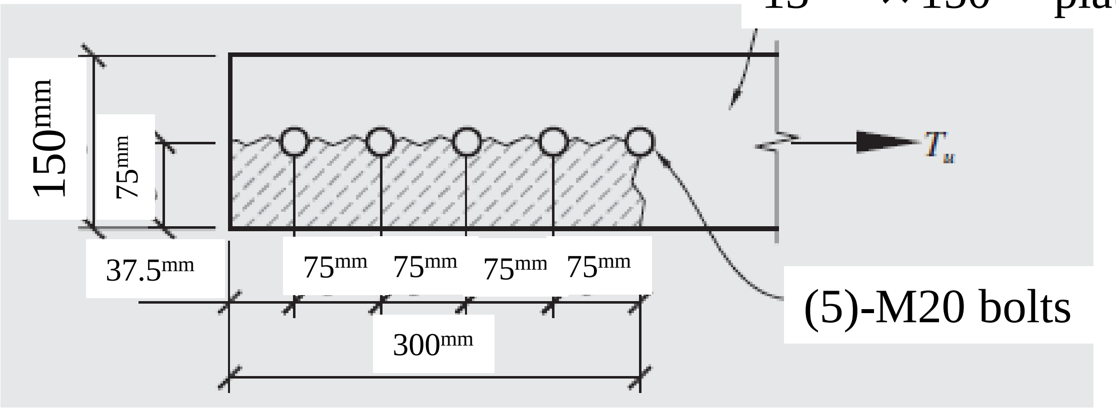
$$A_{nv} = 3900^{mm^2} - (4.5)(20^{mm} + 2^{mm} + 2^{mm})(13^{mm}) = 2496mm^2$$

$$A_{gt} = (75^{mm})(13^{mm}) = 975mm^2$$

$$A_{nt} = A_{gt} - \Sigma A_{holes} = A_{nt} = 975^{mm^2} - (0.5)(20^{mm} + 2^{mm} + 2^{mm})(13^{mm}) = 819mm^2$$

Example 1: Solution

4- Check block shear strength:



$$\phi P_n = \phi\left(0.6F_u A_{nv} + U_{bs} F_u A_{nt}\right) \le \phi\left(0.6F_y A_{gv} + U_{bs} F_u A_{nt}\right)$$

$$\phi P_n = 0.75\left[(0.6)\left(430^{N/mm^2}\right)\left(2496m^{m^2}\right) + (1.0)\left(430^{N/mm^2}\right)\left(819m^{m^2}\right)\right]$$

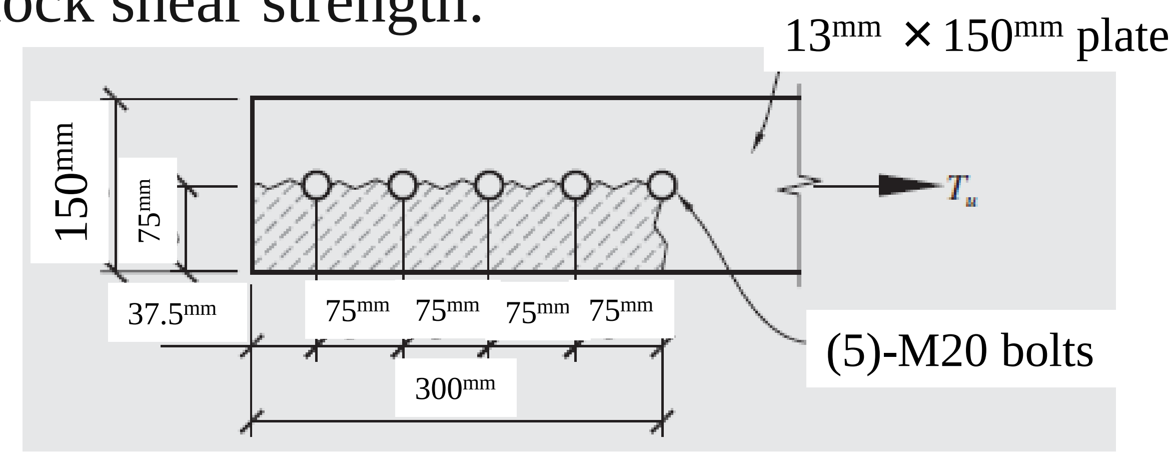
$$\le 0.75\left[(0.6)\left(275^{N/mm^2}\right)\left(3900m^{m^2}\right) + (1.0)\left(430^{N/mm^2}\right)\left(819m^{m^2}\right)\right]$$

$$\phi P_n = 747.1kN \rangle 746.7kN$$

$$\phi P_n = 746.7kN\left(block\ shear\ capacity\right)\rangle P_u = 402.2kN \quad OK$$

Example 1: Solution

4- Check block shear strength:



Use a 13^{mm} × 150^{mm} plate with 5 M20 8.8 bolts

Example 2:

Design a tension member given the following:

- Service loads: P_D = 180 kN, P_L = 295 kN
- Single angle required
- Unbraced length, L = 6 m
- EN10025-S275 steel
- Two lines of four M20 bolts

Example 2: Solution

$P_u = 1.4P_D = (1.4)(180^{kN}) = 252$ kN

$P_u = 1.2P_D + 1.6P_L = (1.2)(180^{kN}) + (1.6)(295^{kN}) = 688$ kN

1- $$A_g \geq \frac{(P_u)}{0.9F_y}$$ (assume slenderness ratio ≤ L/300)

$$A_g \geq \frac{(688000^{N})}{0.9x275^{N/mm^2}} = 2780mm^2$$

Example 2: Solution

2- Sher lag factor, U, is 0.80 for single angles (from ÇYTYE Table 7.1). Alternatively, U may be calculated using l = (3)(75^{mm}) = 225 mm, but an angle size would have to be assumed.

$$A_g \geq \frac{(P_u)}{0.75 F_u U} + \Sigma A_{holes}$$

$$A_g \geq \frac{\left(688000^{N}\right)}{0.75 x 430^{N/mm^2} x 0.80} + (2\ holes)\left(20^{mm} + 2^{mm} + 2^{mm}\right)\left(t^{mm}\right)$$

$$A_{g\ required} = 2667^{mm^2} + 48^{mm} t^{mm}$$ (t is the thickness of the angle)

$$i^{\min} = \frac{L}{300} = \frac{6000^{mm}}{300} = 20mm$$

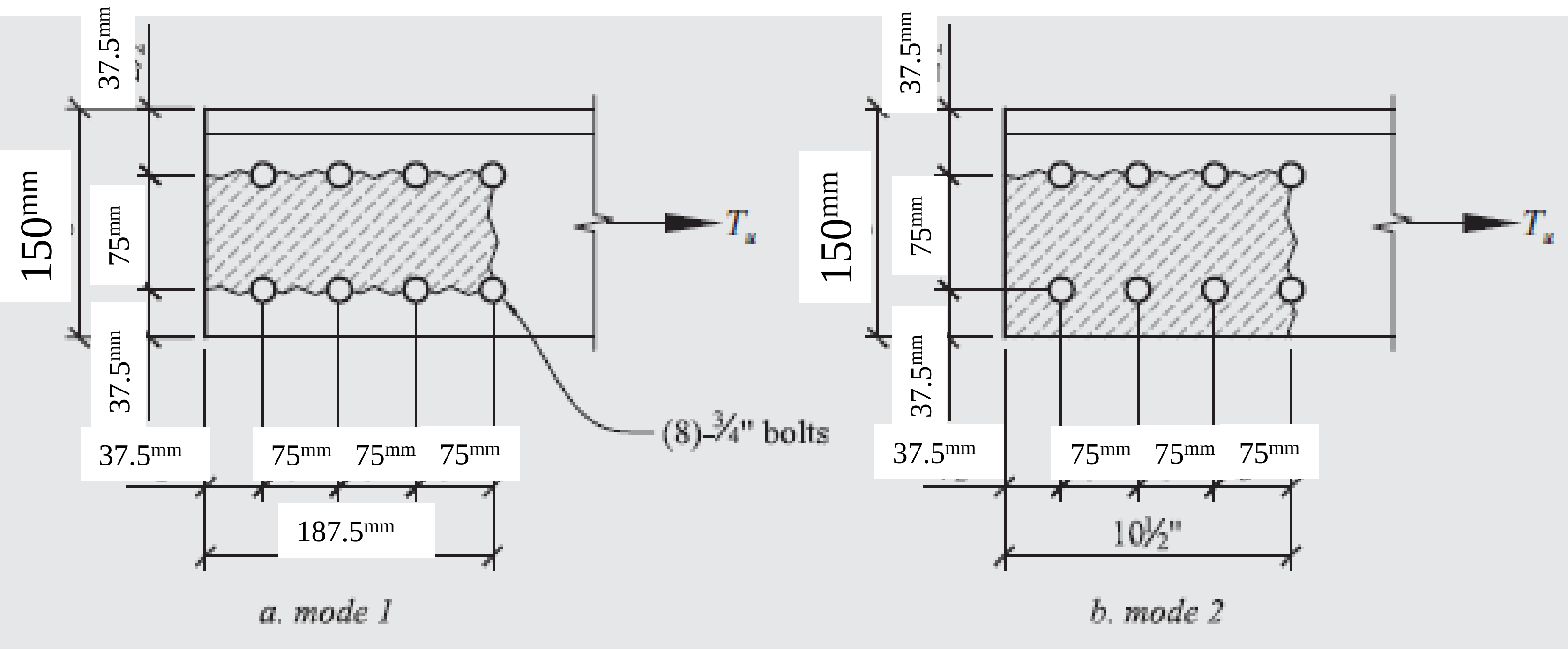
Example 2: Solution

Summary of angle selection:

t (mm)	A_g Required		Selected Angle	i_{min}
	A_g (Step 1)	A_g (Step 2)		
6	2780 mm^2	2955 mm^2	None Worked	-
8	2780 mm^2	3051 mm^2	None Worked	-
10	2780 mm^2	3147 mm^2	None Worked	-
12	2780 mm^2	3243 mm^2	L150×150×12 3480 mm^2	29.4 mm

Example 1: Solution

3- Check block shear strength:



L150 × 150 × 12
M20 bolts

Example 1: Solution

3- Check block shear strength: Mode 1

$$A_{gv} = 2\left(262.5^{mm}\right)\left(12^{mm}\right) = 6300mm^2$$

$$A_{nv} = A_{gv} - \Sigma A_{holes} = 6300^{mm^2} - (3.5)\left(20^{mm} + 2^{mm} + 2^{mm}\right)\left(12^{mm}\right) = 5292mm^2$$

$$A_{gt} = \left(75^{mm}\right)\left(12^{mm}\right) = 900mm^2$$

$$A_{nt} = A_{gt} - \Sigma A_{holes} = A_{nt} = 900^{mm^2} - (1.0)\left(20^{mm} + 2^{mm} + 2^{mm}\right)\left(12^{mm}\right) = 612mm^2$$

$$\phi P_n = \phi\left(0.6F_u A_{nv} + U_{bs} F_u A_{nt}\right) \leq \phi\left(0.6F_y A_{gv} + U_{bs} F_u A_{nt}\right)$$

$$\phi P_n = 0.75\left[(0.6)\left(430^{N/mm^2}\right)\left(5292m^{m^2}\right) + (1.0)\left(430^{N/mm^2}\right)\left(612m^{m^2}\right)\right]$$

$$\leq 0.75\left[(0.6)\left(275^{N/mm^2}\right)\left(6300m^{m^2}\right) + (1.0)\left(430^{N/mm^2}\right)\left(612m^{m^2}\right)\right]$$

$$\phi P_n = 1221.4kN \rangle 977.0kN$$

$$\phi P_n = 977.0kN\left(block\ shear\ capacity\right) \rangle P_u = 688kN \quad OK$$

Example 1: Solution

3- Check block shear strength: Mode 2

$$A_{gv} = 1(262.5^{mm})(12^{mm}) = 3150mm^2$$

$$A_{nv} = A_{gv} - \Sigma A_{holes} = 3150^{mm^2} - (3.5)(20^{mm} + 2^{mm} + 2^{mm})(12^{mm}) = 2142mm^2$$

$$A_{gt} = (75^{mm} + 37.5^{mm})(12^{mm}) = 1350mm^2$$

$$A_{nt} = A_{gt} - \Sigma A_{holes} = A_{nt} = 1350^{mm^2} - (1.5)(20^{mm} + 2^{mm} + 2^{mm})(12^{mm}) = 918mm^2$$

$$\phi P_n = \phi(0.6F_u A_{nv} + U_{bs} F_u A_{nt}) \leq \phi(0.6F_y A_{gv} + U_{bs} F_u A_{nt})$$

$$\phi P_n = 0.75[(0.6)(430^{N/mm^2})(2142m^{m^2}) + (1.0)(430^{N/mm^2})(918m^{m^2})]$$

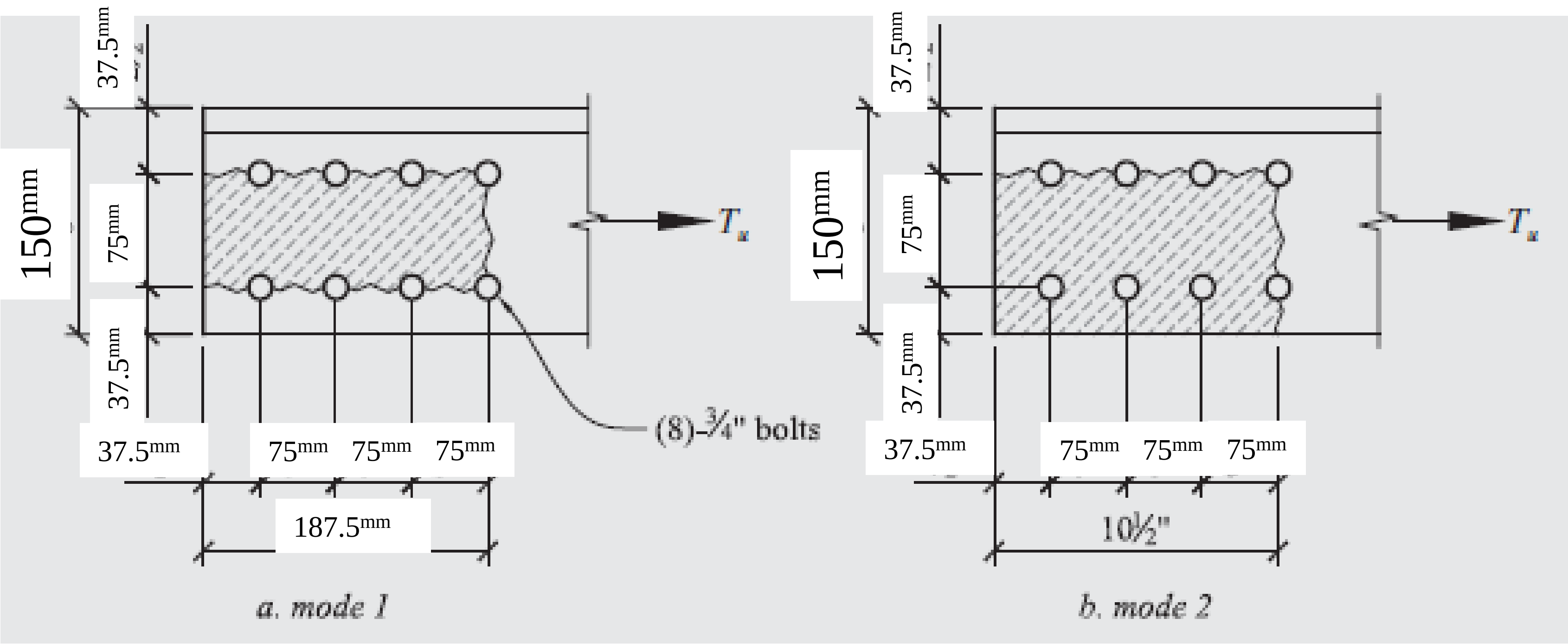
$$\leq 0.75[(0.6)(275^{N/mm^2})(3150^{mm^2}) + (1.0)(430^{N/mm^2})(918m^{m^2})]$$

$$\phi P_n = 710.5kN \rangle 587.2kN$$

$$\phi P_n = 587.02N(block\ shear\ capacity) \langle P_u = 688kN \quad NOT\ OK$$

Example 1: Solution

3- Increase the angle size and check block shear strength of Mode 2:



L150 × 150 × 14

M20 bolts

Example 1: Solution

3- Check block shear strength: Mode 2

$$A_{gv} = 1\left(262.5^{mm}\right)\left(14^{mm}\right) = 3675mm^2$$

$$A_{nv} = A_{gv} - \Sigma A_{holes} = 3675^{mm^2} - (3.5)\left(20^{mm} + 2^{mm} + 2^{mm}\right)\left(14^{mm}\right) = 2499mm^2$$

$$A_{gt} = \left(75^{mm} + 37.5^{mm}\right)\left(14^{mm}\right) = 1575mm^2$$

$$A_{nt} = A_{gt} - \Sigma A_{holes} = A_{nt} = 1575^{mm^2} - (1.5)\left(20^{mm} + 2^{mm} + 2^{mm}\right)\left(14^{mm}\right) = 1071mm^2$$

$$\phi P_n = \phi\left(0.6F_u A_{nv} + U_{bs} F_u A_{nt}\right) \le \phi\left(0.6F_y A_{gv} + U_{bs} F_u A_{nt}\right)$$

$$\phi P_n = 0.75\left[(0.6)\left(430^{N/mm^2}\right)\left(2499m^{m^2}\right) + (1.0)\left(430^{N/mm^2}\right)\left(1071m^{m^2}\right)\right]$$

$$\le 0.75\left[(0.6)\left(275^{N/mm^2}\right)\left(3675^{mm^2}\right) + (1.0)\left(430^{N/mm^2}\right)\left(918^{mm^2}\right)\right]$$

$$\phi P_n = 829.0kN \rangle 800.2kN$$

$$\phi P_n = 800.2N\left(block\ shear\ capacity\right) \rangle P_u = 688kN \quad OK$$