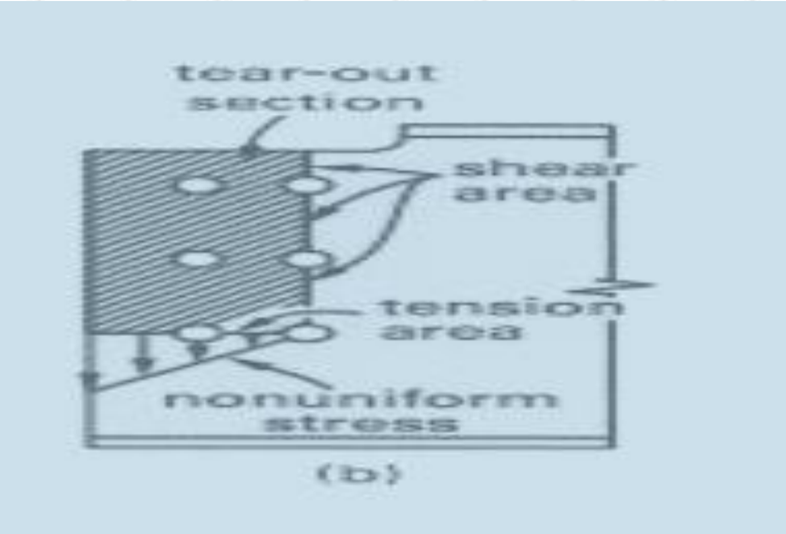
EXAMPLE 10.8
Block Shear Strength

Goal: Determine the block shear design strength and allowable strength for a coped beam.

Given: A coped W16×40 A992 beam end is shown in Figure 10.24. Assume that the beam has standard holes for 5/8 in. bolts.



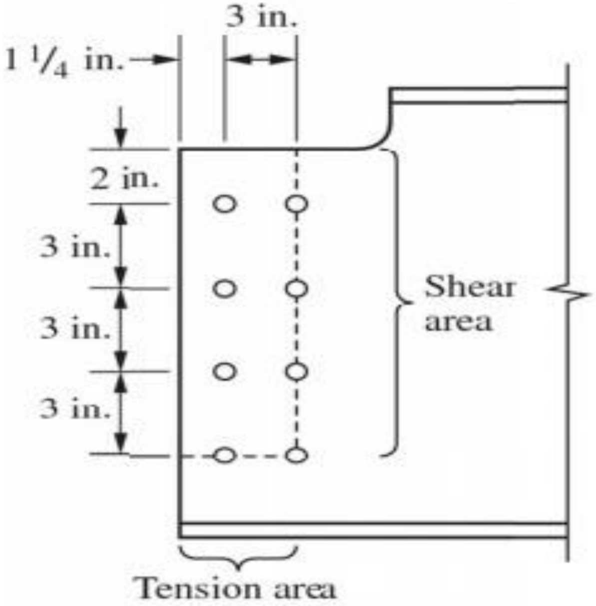


Figure 10.24 Coped Beam End for Example 10.8

Case -2

Non uniform stress

near edge ⇐ Shear Force

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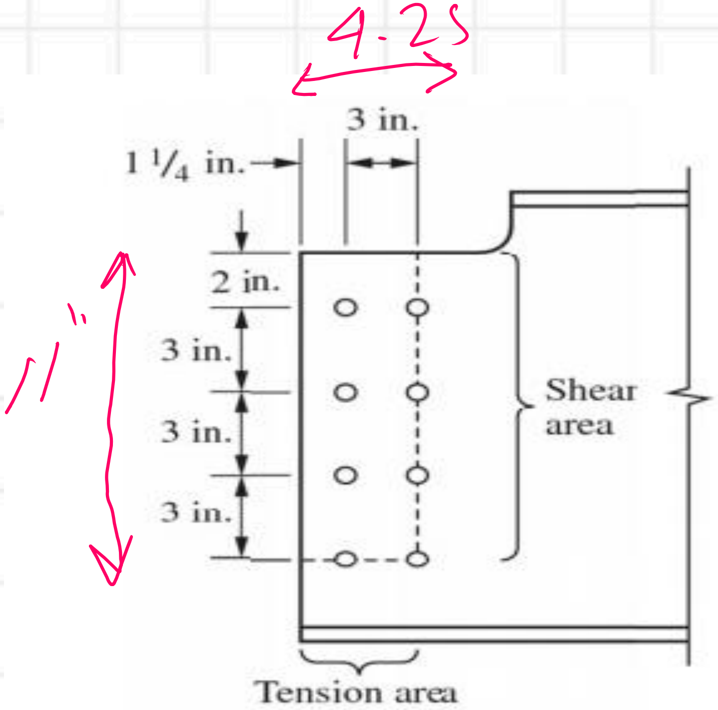
This is Case #2, there are two Lines of bolts



$W_{16} \times 40$ $\quad d_b = \frac{5}{8} \Rightarrow d_h = d_b + \frac{1}{8}$"

$d_h = \frac{5}{8}" + \frac{1}{8}" = \frac{6}{8}"$

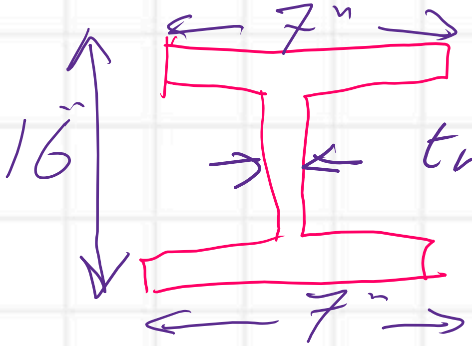
A992 $\quad F_y = 50$ ksi

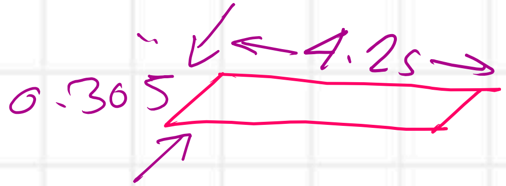
$F_u = 65$ ksi

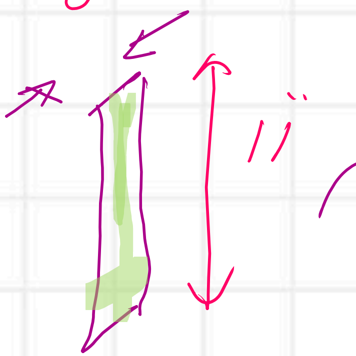
Figure 10.24 Coped Beam End for Example 10.8

Gross areas

The section of $W_{16} \times 40$ is as follows







shear

Tension section

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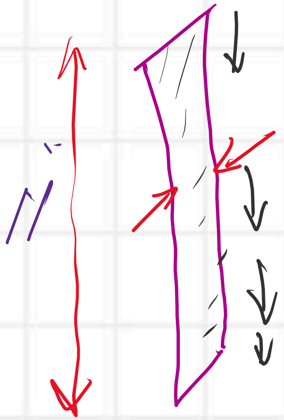
Shear yielding

$F_y = 50\ ksi$

$\Rightarrow t_w = 0.305''$



$A_{gv} = t_w \cdot h_w = 0.305(11) = 3.355\ inch^2$ } $d_h = \frac{6}{8}''$

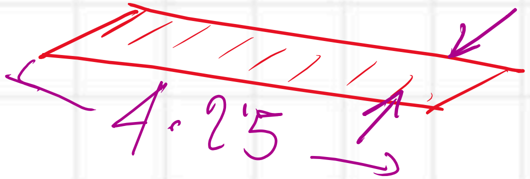
shear stress $= 0.60\ F_y$
$= 0.6(50) = 30\ ksi$

shear yielding $= (0.60\ F_y) A_{gv} = 0.60(50)(3.355) = 100.65$ kips

Case of tension rupture

A_{gt} : gross area for tension) No. of holes $= 1.5$

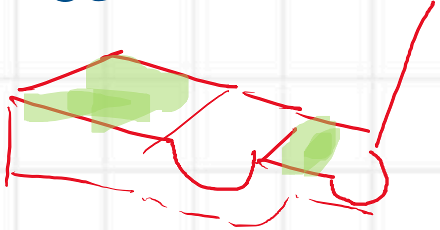
$F_U = 65\ ksi$



$t_w = 0.305''$

$A_{gt} = 4.25(0.305)$

$A_{gt} = 1.296\ inch^2$

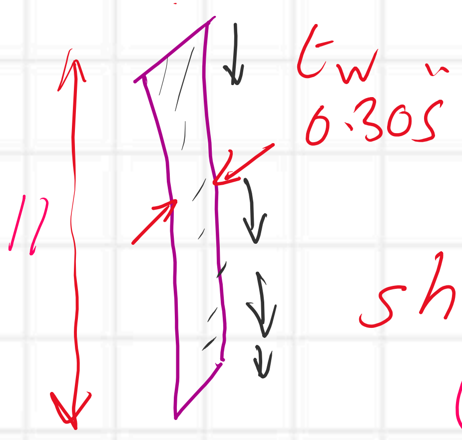


$A_{nt} = A_{gt} - 1.5\, d_h \cdot t_w$
$= 1.296 - 1.5\left(\frac{6}{8}\right)0.305$
$= 0.9531\ inch^2$

$UBS = \frac{1}{2} \rightarrow UBS\ A_{nt}\ (0.60) F_U = (0.9531)\frac{1}{2}(65) = 30.976$ k

Case of shear rupture

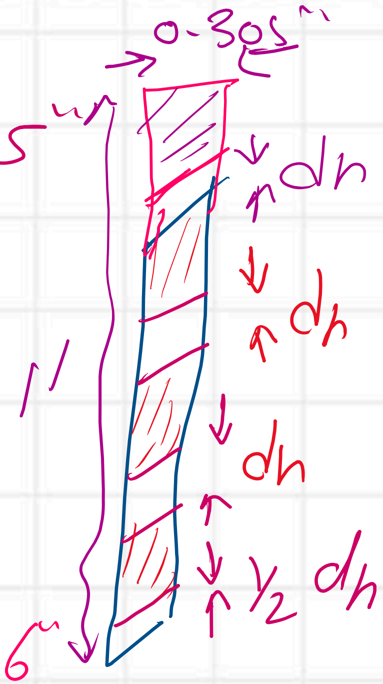
$F_u = 65$ ksi



$A_{gv} = 3.55$ inch^2

$A_{nv} = 2.554$ inch^2

$\Rightarrow t_w = 0.305''$



shear stress $= (0.60)(F_u) = 0.6(65) = 39$ ksi

$(0.60 F_u)(A_{nv})$

shear rupture $= (39)(2.554) = 99.60$ kips

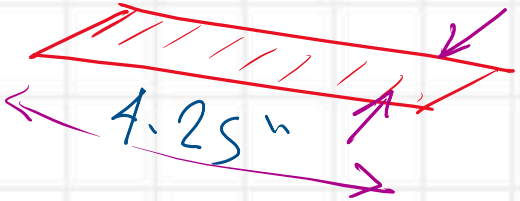
$d_h = \frac{6}{8}''$

No. of holes $= 3.50$

$A_{nv} = 3.55 - (0.305)3.5\left(\frac{6}{8}\right)$

Case of tension rupture

A_{nt} value



$(t_w = 0.305'')$

$A_{gt} = 1.296$ inch^2, $A_{nt} = 0.9531$ inch^2

$F_{nt} = 30.976$ kips

$U_{bs} = \frac{1}{2}$

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Shear yieLding = 100.65 kips

Shear rupture = 99.60 kips

$A_{nt} = 0.9531$

Tension Rupture

$U_{BS}(F_U)A_{nt} = \frac{1}{2}(0.9531)(65) = 30.976^{K}$

Tension rupture = 30.976 kips

Total : $0.60 F_y A_{gv} + U_{BS} . A_{nt} . F_U$

Failure is Controlled by shear rupture + Tension rupture

$R_{n_1} = 100.65 + 30.976 = 131.626^{K}$

$0.60 F_U A_{nv} + U_{BS} A_{nt} . F_{Ult}$

$R_{n2} = 99.60 + 30.976 = 130.576^{K}$

$\phi R_1 = [0.75(131.626)]$

$\phi = 0.75$

$\phi R_2 = 0.75[130.576] = 97.93\ K$

$\simeq 98$ kips

$\phi R_1 = 98.720$ kips

$\phi R = \min(98.720, 98] = 98\ K$ min. Value

$\boxed{LRFD}$

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Shear yieLding = 100.65 kips

$A_{nt} = 0.9531$

Tension rupture

$U_{Bs} A_{nt} F_U \left(\frac{1}{2}(0.9531) \cdot 65 = 30.976^K\right.$

Total : $0.60 F_y A_{gv} + U_{BS} \cdot A_{nt} \cdot F_U$

$R_{n_1} = 100.65 + 30.976 = 131.626^K$

Shear rupture = 99.60 kips

Tension rupture = 30.976 kip's

Failure is Controlled by shear rupture + Tension rupture

$0.60 F_U A_{nv} + U_{Bs} A_{nt} \cdot F_{Ult}$

$R_{n2} = 99.60 + 30.976 = 130.576^K$

ASD value $\Omega = 2$

$\frac{1}{\Omega} R_1 = \frac{1}{2}(131.626)$

$= 65.813$ kips

$\frac{1}{\Omega} R_2 = \frac{1}{2}(130.576) = 65.288K$

$\frac{1}{\Omega} R = \min(65.813, 65.288) = 65.288$ kips $\simeq 65.30$ kips

ASD

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