Engineering Instruction

Bracing Connection Design Forces

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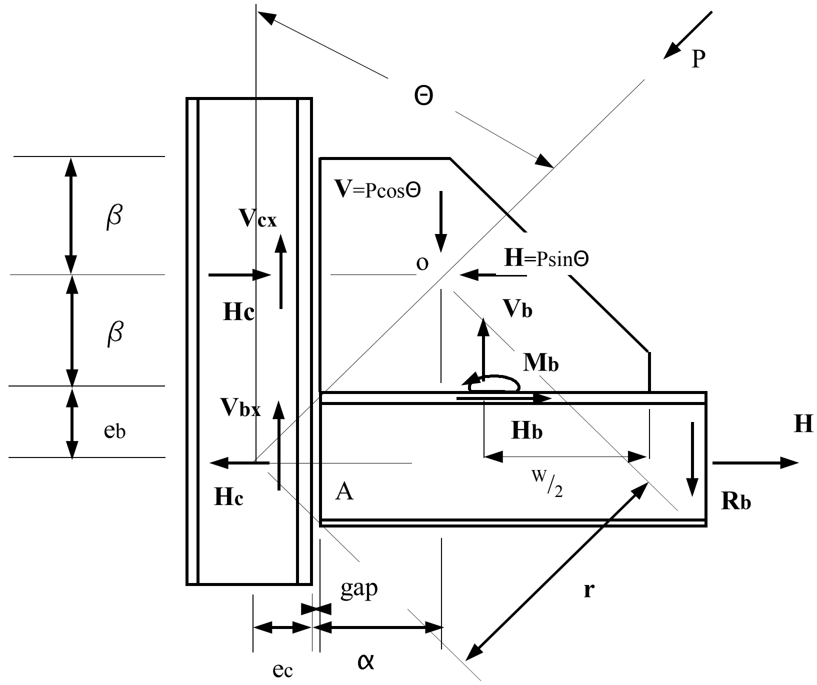


H, the horizontal component of the brace force acting at the column and beam centerlines is resisted by an axial force in the beam and/or a transfer force acting through the column at the beam centerline. (not shown)

V, the vertical component of the brace force acting at the column centerline is resisted at the face of the column by a gusset force and a beam force acting together such that;

$$V = V_c + V_b$$

The moment due to eccentricity e_c is resisted by an assumed couple Hc acting at beam and gusset plate centerlines such that;

$\mathbf{V \times e_c}$ **must equal** $\mathbf{H_c \times (e_b + \beta)}$

$$\text{ie } \mathbf{H_c} = V\left({}^{e_c}/_{e_b+\beta}\right) = P\cos\Theta\left({}^{e_c}/_{e_b+\beta}\right) = P\left({}^{e_b+\beta}/_{r}\right)\left({}^{e_c}/_{e_b+\beta}\right) = \mathbf{\left({}^{e_c}/_{r}\right) P}$$

$$\text{and } \mathbf{H_b} = H - H_c = P\sin\Theta - \left({}^{e_c}/_{r}\right)P = P\left[{}^{(e_c+\alpha)}/_{r} - \left({}^{e_c}/_{r}\right)\right] = \mathbf{\left({}^{\alpha}/_{r}\right) P}$$

P, e_c, e_b & Θ are Given

$\tan\Theta = (e_c + \alpha)/(e_b + \beta)$

$\alpha = (e_b + \beta)\tan\Theta - e_c$

β = (gusset height)/2

W = width of gusset plate

$r = \sqrt{(e_c + \alpha)^2 + (e_b + \beta)^2}$

Gap = gusset plate setback from column flange (½ in typical)

If V and R_b are distributed, considering brace T or C , so that all bolts in the column flange have equal vertical loads;

$$V/\text{bolt} = (V + R_b) / (\text{ total number of bolts })$$

$\mathbf{V_{cx}}$ = **V/bolt x (number of bolts gusset to column)**

$\mathbf{V_{bx}}$ = **V/bolt x (number of bolts beam to column)**

$\mathbf{V_b = V - V_{cx}}$

then from statics

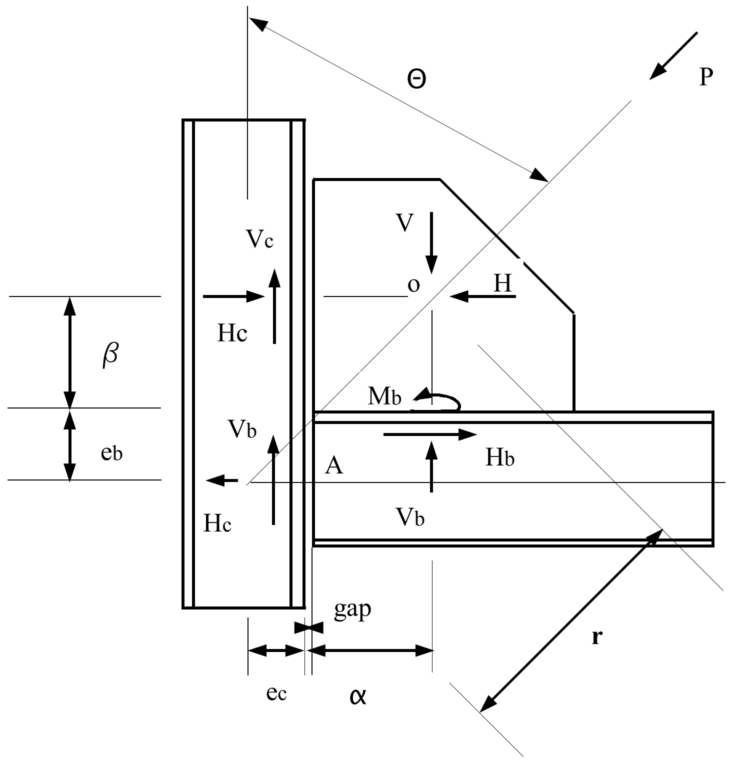
$$\mathbf{M_b = V_{cx}\left({}^{W}/_{2} + gap\right) - V\left({}^{W}/_{2} + gap - \alpha\right) - \beta\left(H - H_c\right)}$$

Notes:
1. Manipulating the gusset plate width to make Mb zero (see Uniform Force Method below) for brace in tension will not make it zero for brace in compression .
2. The space required for the brace to gusset connection usually dictates minimum gusset width and the gusset should be made longer if that will make a single pass (5/16) fillet weld work.
3. Gusset plate thickness should be comparable to web thickness considering the grades of steel used.
4. Beam webs should be filled with bolts to make compact and economical connections.
5. Connection angle thickness will often be controlled by prying on the outstanding leg from H_c.
6. For brace connection to column web let e_c and H_c equal zero.

The Uniform Force Method

1. Ideal Geometry $M_b = 0$

Consider a FBD of the Gusset Plate
And the geometry as shown



1. Moment about o

$$V_c \alpha = H_b \beta$$
or
$$H_b = (V_c \alpha) / \beta$$

2. $\Sigma H = 0$

$$H_b + H_c = H = \left[(e_c + \alpha) / r \right] P$$

3. $\Sigma V = 0$

$$V_b + V_c = V = \left[(e_b + \beta) / r \right] P$$

Consider a FBD of the Beam

4. $\Sigma M_A = 0$

$$V_b \alpha = H_b e_b$$
or
$$V_b = (H_b e_b) / \alpha$$

Substituting 4 into 3:

$$(H_b e_b) / \alpha + V_c = \left[(e_b + \beta) / r \right] P$$

Substituting From 1:

$$V_c (\alpha / \beta) (e_b / \alpha) + V_c = (e_b + \beta)(P / r)$$

$$V_c (e_b / \beta + 1) = (e_b + \beta)(P / r)$$

So:

$$\mathbf{V_c} = (e_b + \beta)(P / r) \; / \; (e_b / \beta + 1) = (\beta / r) P$$

$$\mathbf{V_b} = \left[(e_b + \beta) / r \right] P - V_c = \left[(e_b + \beta - \beta) / r \right] P = (e_b / r) P$$

$$\mathbf{H_b} = V_b \alpha / e_b = (e_b / r) P \alpha / e_b = (\alpha / r) P$$

$$\mathbf{H_c} = \left[(e_c + \alpha) / r \right] P - H_b = \left[(e_c + \alpha) / r - \alpha / r \right] P = (e_c / r) P$$

Note all are functions of geometry only

Note
P, e_c, e_b & Θ are Given

β = (gusset plate height) / 2

Choosing β defines α for $M_b = 0$

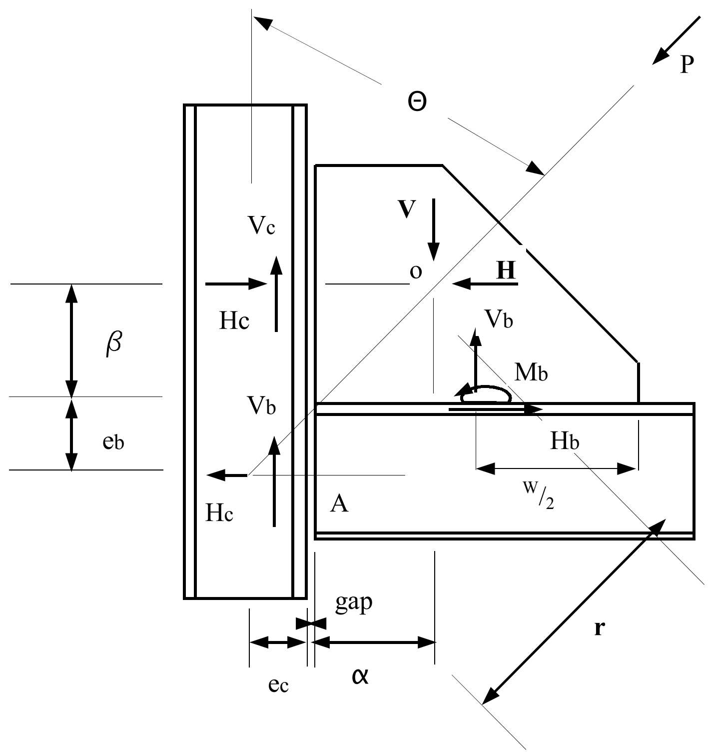
α = (gusset plate width) / 2 + gap

$$r = \sqrt{(e_c + \alpha)^2 + (e_b + \beta)^2}$$

The Uniform Force Method

2. Non Ideal Geometry $M_b \neq 0$

Let all remain as in case 1 except $W/2 \neq \alpha$



Consider a FBD of the Gusset Plate
And the geometry as shown

1. Moment about o

$V_c\,\alpha = H_b\,\beta + V_b(W/2 - \alpha) + M_b$

but from case 1

$V_c\,\alpha = H_b\,\beta$

so

$\mathbf{M_b = V_b(W/2 - \alpha)}$

And as in Case 1

$\mathbf{V_c = (\beta/r)\,P}$

$\mathbf{V_b = (e_b/r)\,P}$

$\mathbf{H_b = (\alpha/r)\,P}$

$\mathbf{H_c = (e_c/r)\,P}$

Note all are functions of geometry only

Note
P, e_c, e_b & Θ are Given

β = (gusset plate height) / 2

Choosing β defines α for $M_b = 0$

In this case

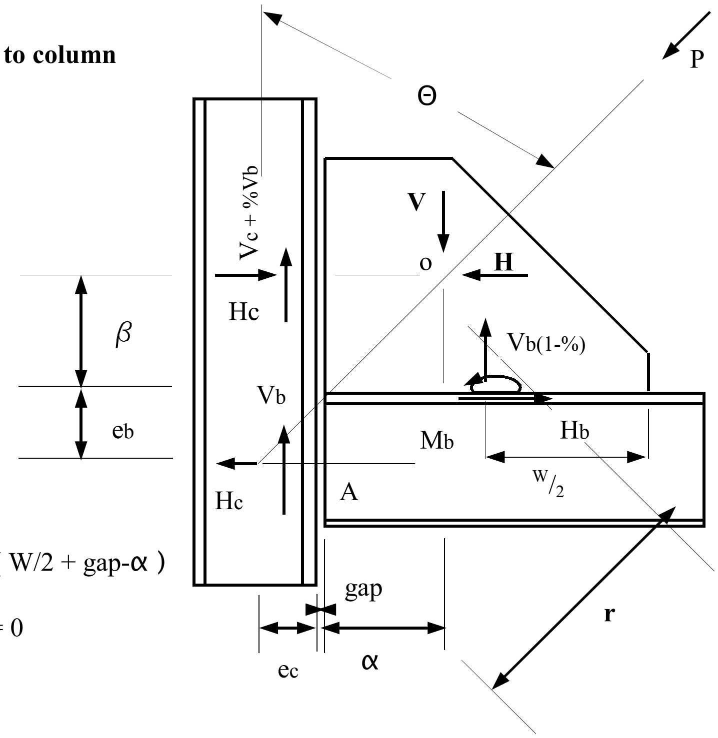
$\alpha \neq$ (gusset plate width) / 2 + gap

$r = \sqrt{(e_c + \alpha)^2 + (e_b + \beta)^2}$

The Uniform Force Method

3. Non Ideal Geometry

$M_b \neq 0$ and part of V_b transferred to column



Let all else remain as in case 2

Consider a FBD of the Gusset Plate
And the geometry as shown

1. Vertical Force at center of gusset =

   $V_b(1-\%) = V - (V_c + \%V_b)$

   Moment at center of gusset =

   $H_c\beta + (V_c + \%V_b)(W/2 + gap) - H\beta - V(W/2 + gap - \alpha)$

   When % = 0 and $(W/2 + gap) = \alpha$ $\quad M = 0$

   ie $0 = H_c\beta + V_c\alpha - H\beta$

   Subtracting

   $\mathbf{M} = V_c(W/2 + gap - \alpha) + \%V_b(W/2 + gap) - V(W/2 + gap - \alpha)$

   $\mathbf{= \%V_b(W/2 + gap) - V_b(W/2 + gap - \alpha)}$

Note
P, e_c, e_b & Θ are Given

β = (gusset plate height) / 2

Choosing β defines α for $M_b = 0$

In this case

$\alpha \neq$ (gusset plate width) / 2 + gap

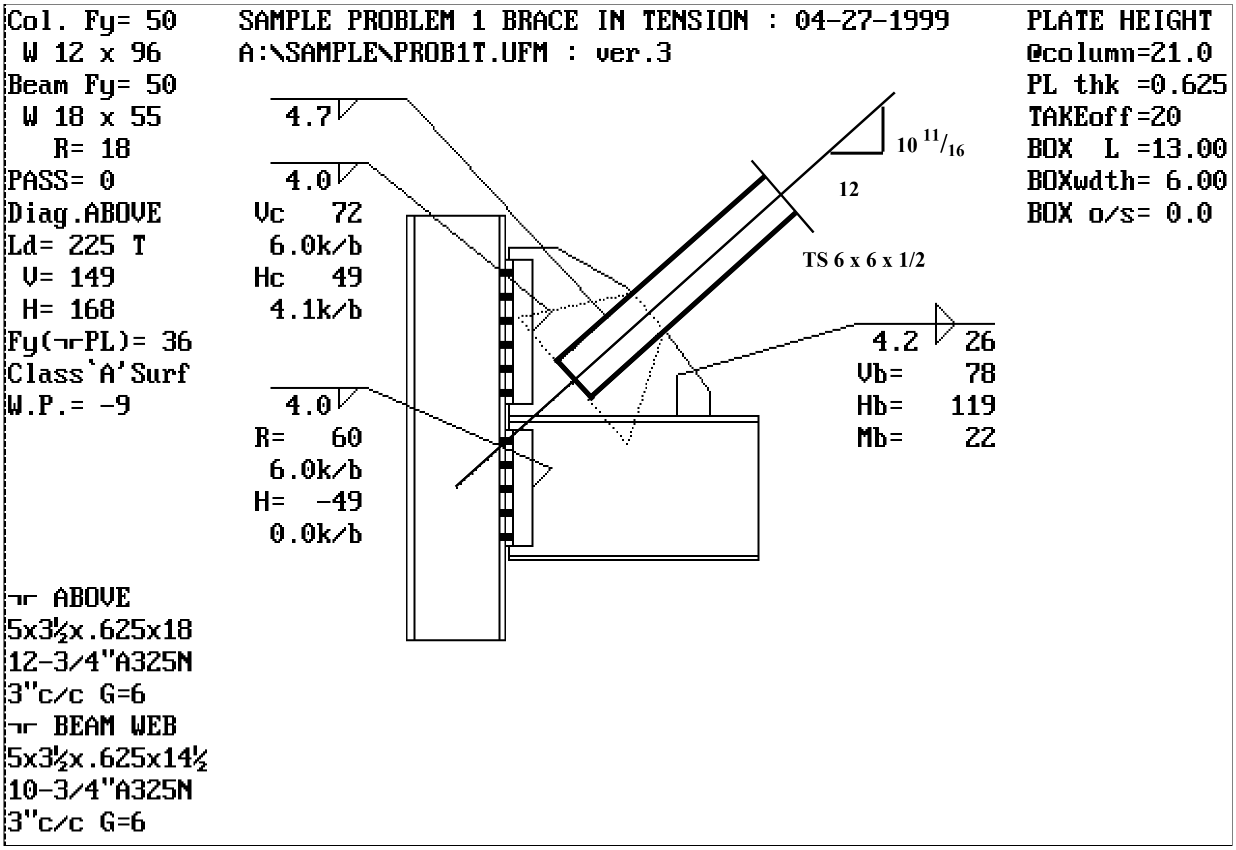
$r = \sqrt{(e_c + \alpha)^2 + (e_b + \beta)^2}$

V_{col} = $\mathbf{(\beta/r)\,P + \%\,(e_b/r)\,P}$

V_b = $\mathbf{(1 - \%)\,(e_b/r)\,P}$

H_b = $\mathbf{(\alpha/r)\,P}$

H_c = $\mathbf{(e_c/r)\,P}$



Column depth	= 12.71 in.	e_c	= column depth / 2 = 6.355 in
Web thickness	= .55 in.	e_b	= from top of flange to W.P. = 9
		α	$= (e_b + \beta)\tan\Theta - e_c = 15.53$
Beam depth	= 18.11 in.	β	= 10.5 in. = plate height @ col. / 2
Flange width	= 7.53 in.	$\tan\Theta$	= H / V = 168/149.7
Flange thickness	= .63 in.	Θ	= 48.296
Web thickness	= .39 in.	r	$= \sqrt{(e_c + \alpha)^2 + (e_b + \beta)^2} = 29.31$
k	= 1.3125	gap	= .5 in.
k_1	= .8125	W	= gusset plate width = 26 in.
Connection material Fy	= 36	Weld sizes shown in 16ths	
Pass through load	= 0	Bolted faying surfaces class A	

Bracing Connection Design Forces

$\mathbf{H_c} = \left(\frac{e_c}{r}\right) \mathbf{P} = (6.355 / 29.31)(225) =$ **48.78 kips**

$\mathbf{H_b} = \left(\frac{\alpha}{r}\right) \mathbf{P} = (15.53 / 29.31)(225) =$ **<u>119.21</u> kips**

Hc + Hb = 167.99 kips = H = 168 KIPS

V/bolt = (149.7 - 18) / 22 = 5.986 kips / bolt

$\mathbf{V_{cx}}$ = 5.986 x 12 = **71.83 kips** $\mathbf{V_{bx}}$ = 5.986 x 10 = **59.86 kips**

$\mathbf{V_b = V - V_{cx}}$ = (149.7 – 71.83) = **77.87 kips**

$$\mathbf{M_b = V_{cx}\left(\frac{W}{2} + gap\right) - V\left(\frac{W}{2} + gap - \alpha\right) - \beta\left(H - H_c\right)}$$

= 71.83(26/2 + .5) -149.7(26/2 + .5 - 15.53) - 10.5(168 – 48.78)

= 969.705 + 303.891 – 1251.81 = **21.79 in-kips**

Stresses along edge of gusset plate at beam flange

Gusset plate thickness = .625 inch.
Shear force = f_v = Hb / width of plate = 119.21 / 26 = 4.585 kips / inch
Normal force = f_n = Vb / width of plate = 77.87 / 26 = 2.995 kips / inch
and f_b = Mb * 6 / (plate width)^2 = (21.79 x 6) / (26)^2 = 0.193 kips / inch

Shear stress in gusset = fv / thickness = 4.585 / .625 = 7.336 ksi MUST BE <= .4Fy =.4 x 36 = 14.4 ksi

Normal stress in gusset = (fn + fb) / thick = (2.995 + .193) / .625 = 5.101 ksi MUST BE <= .6Fy = 21.6 ksi

Weld

Peak force on weld = $f_{peak} = \sqrt{(fn + fb)^2 + fv^2} = \sqrt{(2.995 + 0.193)^2 + (4.585)^2}$ =5.584 kips / inch

Average force on weld = $f_{avg} = \sqrt{\left(\frac{fn + fb}{2}\right)^2 + fv^2} = \sqrt{[(2.995 + 0.193) / 2]^2 + (4.585)^2}$ = 5.530 kips / inch

Required weld size is the <u>GREATER</u> of f_{peak}/(.928 * 2 welds) = 5.584 / (.928 * 2) = 3.0/16ths

or 1.4 f_{avg}/(.928 * 2 welds) = (1.4 x 5.53) / (.928* 2) = **4.2/16ths**

(USE 5/16 th FILLET)

Normal stresses on beam web at end of gusset plate

Normal force at either end of gusset
= (Vb / Length) + or - (Mb * 6 / Length^2)

where Length = Width of Gusset + 2.5 x k = 26 +2.5 x 1.3125 =29.281 in.

= 77.87 / 29.281 - (21.79 x 6 / 29.281^2) = 2.507 kips/inch @ LEFT END

= 77.87 / 29.281 + (21.79 x 6 / 29.281^2) = 2.811 kips/inch @ RIGHT END

Maximum normal stress is <u>GREATER</u> of above / web thickness

= 2.811 / .39 = 7.209 ksi MUST BE <= .66 Fy =.66 x 50 = 33 ksi

Crippling of web under compression from gusset

Localized compression load on web due to gusset forces above = 0 kips
(tension at both ends of gusset for this loading condition)

check is not applicable

Horizontal force on beam from gusset

Horizontal. shear capacity of web plus axial capacity of flange must be greater than **H_b**

$.4F_y$ x (Gusset Width + 2.5 x k) x Web Thick + $.6F_y$ x Area of Flange must be greater than 119.21 kips

.4 x 50 x (26 + 2.5 x 1.3125) x.39 + .6 x 50 x 7.53 x .63 =228.39 + 142.32 = 370.71 kips

which is greater than 119.21 kips

Shear in column web

Column web shear capacity must be greater than **H_c**

.4Fy x Column depth x column web thickness must be greater than or equal to 48.78 kips

.4 x 50 x 12.71 x .55 = 139.81 kips which is greater than 48.78 kips

IF THERE IS A BRACING CONNECTION ON THE FAR SIDE OF THE COLUMN,
Hc FROM THAT CONNECTION MAY BE ADDITIVE TO THE ABOVE.
THE TOTAL MUST BE CHECKED AGAINST THE COLUMN CAPACITY.
WEB DOUBLERS MAY BE REQUIRED

Welded TS 6 x 6 x 1/2 diagonal

Welds on longitudinal sides of slotted TS 6 x 6 x 1/2, both sides of gusset plate, 4 welds in all
Length of each fillet weld = 13 in. (MINIMUM LENGTH = TS WIDTH = 6 in.)
Total length of fillet weld = 4 x 13 = 52 in.

Required fillet weld size = brace load / (0.928 x total length of weld)
= 225 / (0.928 x 52) = 4.66 16ths (USE 5/16 TH FILLET)

Whitmore width = width of TS + 2tan30 x (length of each weld) = 6 + 2tan30 (13) = 21.01 in.

Tension capacity of whitmore section = whitmore width x gusset thickness x .6 F_y
= 21.01 x .625 x .6 x 36
= 283.64 kips > 225 kips **OK**

Block shear capacity of gusset plate around weld and end of TS 6 x 6 x 1/2 is greater of

[(A_v x .4 Fy) + (A_t x .5F_u)] = [(2 x 13 x .625 x .4 x 36) + (6 x .625 x .5 x 58)] = 234 + 108.75 = 342.75 kips
or
[(A_v x .3 F_u) + (A_t x .6Fy)] = [(2 x 13 x .625 x .3 x 58) + (6 x .625 x .6 x 36)] = 282.75 + 81 = 363.75 kips

But not more than 4 x length of each weld x gusset thickness x $.4F_y$ = 4 x 13 x .625 x .4 x 36 = 468 kips

Welds angles to gusset plate

Design is by vector analysis

Horizontal length of weld = angle leg size – 1/2 in. gap = $3^1/_2 - {}^1/_2 = 3$ in.

Vertical length of weld = angle length = 18 in.

Total weld length = 3 + 18 + 3 = 24 in.

Polar moment of inertia of weld profile = $(2 \times 3 + 18)^3 / 12 - (3^2 (3 + 18)^2 / (2 \times 3 + 18)$

= 1152 – 165.375 = 986.625 in^3

Centroid of weld from vertical leg = $3^2 / (2 \times 3 + 18) = 0.375$ in.

Moment on weld = V_{cx} x (angle leg size – centroid distance) = 71.83 x (3½ -.375)

= 224.77 in.-kips

Tensile force at edge of plate = V_{cx} / total length + moment x (horizontal weld length – centroid distance) / I_p

fn = 71.83 / 24 + 224.77x (3 – .375) / 986.625 = 2.99 + 0.60

= 3.59 kips / in.

Shear force at edge of plate = H_c / total weld length + moment x (vertical length / 2) / I_p

fv = 48.78 / 24 + 224.77 x (18/2)/ 986.625

= 2.03 + 2.05 = 4.08 kips / in.

Required fillet weld size = $\sqrt{(3.59)^2 + (4.08)^2} / (0.928 \times 2) = 5.43 / (0.928 \times 2)$

= 2.93 16ths (**use ¼ in**. minimum size for 5/8 material)

Shear stress in gusset plate @ toe of angles = V_{cx} / total length – (moment x centroid distance) / I_p

fv′ = 71.83 /24 - (224.77 x .375) / 986.625

= 2.99 - 0.08 = 2.90 Kips / in.

Minimum required gusset plate thickness

fn / .6Fy = 3.59 / .6 x 36 = 0.166 in.

fv / .4Fy = 4.08 / .4 x 36 = 0.283 in.

fv′ / .4Fy = 2.90 / .4 x 36 = 0.201 in. (**OK** 5/8 in plate used)

Bolts and angles (gusset plate to column)

12 - ¾ dia. A325N @ 3 inches c/c, gage = 6

Tension per bolt = 48.78 / 12 = 4.06 kips

Shear per bolt = 71.83 / 12 = 5.98 kips

Check prying on column flange (ref. AISC 9th edition pg 4-90)

b = 2.725 in.

a = 2.3125 in. ≤ 1.25 x (b col. flg.) = 1.25 x (bf – gage) / 2

=1.25 x (12.16 – 6) /2 = 3.85 in. OK

≤ 1.25 x (b angles)

= 1.25 x (2 x angle leg + plate – gage) / 2

= 1.25 x (2 x 5 + .625 – 6) /2

= 2.89 in. OK

p = 3 in. ≤ 5 x bolt diameter = 3.75 in. OK

Bc = 4.06 kips; ie tension plus prying

Permissible Shear on N type bolts = 9.07 kips > 5.98 kips Ok

Permissible Shear on X type bolts = 12.96 kips > 5.98 kips Ok

Independent of prying, the permissible shear/bolt in slip critical connections = 6.42 kips > 5.98 kips Ok

Required minimum bolt spacing in column flange = 2 x (shear / bolt) / (Fu x $t_{fcolumn}$) + (bolt dia) / 2
= 2 x (5.98) / (65 x 0.900) + .75 / 2
= 0.204 +.375 = 0.579 in. OK

Bearing stress of bolts on column flange = (shear / bolt) / ($t_{fcolumn}$ x bolt dia)
= 5.98 / (0.900 x .75)
= 8.85 ksi < 1.2 F_u = 1.2 x 65 = 78 ksi OK

Check prying on angles b = 2.0625 inch.
a = as above
p = 3 in. $\leq$ 5 x bolt diameter = 3.75 in.

Bc = 4.66 kips; ie tension plus prying
Permissible Shear on N type bolts = 9.00 kips > 5.98 kips Ok
Permissible Shear on X type bolts = 12.87 kips > 5.98 kips Ok
Independent of prying, the permissible shear/bolt in slip critical connections = 6.42 kips > 5.98 kips Ok

Required minimum bolt spacing in angles = 2 x (shear / bolt) / (Fu x t_{angle}) + (bolt dia) / 2
= 2 x (5.98) / (58 x 0.625) + .75 / 2
= 0.330 +.375 = 0.705 in. OK

Bearing stress of bolts on angles = (shear / bolt) / (t_{angle} x bolt dia)
= 5.98 / (0..625 x .75)
= 12.75 ksi < 1.2 F_u = 1.2 x 58 = 69.6 ksi OK

Bolt end distance on angle = 1.5 in. > 1.5 x bolt dia. = 1.5 x .75 =1.125 in. OK

Angle shear Total shear = $\sqrt{V_{cx}^2 + H_c^2}$ = $\sqrt{(71.83)^2 + (48.78)^2}$
= 86.82 kips
Net area = 2 x t (angle length – no. rows x (bolt dia +1/16))
= 2 x .625 x (18 – 6 x (3/4 + 1/16))
= 16.406 in.^{2}
Gross area = 2 x t x angle length = 2 x .625 x 18
= 22.50 in.2
Net shear = 86.82 / 14.406
= 5.29 ksi < .3F_u = .3 x 58 = 17.4 ksi OK
Gross shear = 86.82 / 22.5
= 3.86 ksi < .4F_y = .4 x 36 = 14.4 ksi OK

Bolts and angles (beam web to column)

10 - ¾ dia. A325N @ 3 inches c/c, gage = 6
Tension per bolt = 0 kips (for brace in tension loading condition)
Shear per bolt = 5.98 kips (bolt loads were made equal in gusset and beam)

No prying check required for this loading condition
Permissible Shear on N type bolts = 9.28 > 5.98 kips Ok
Permissible Shear on X type bolts = 13.26 > 5.98 kips Ok
Permissible Shear on slip critical bolts = 7.51 > 5.98 kips Ok

Bolt spacing and bearing on column flange same as at gusset plate.
Bolt spacing and bearing angle same as at gusset plate.

Bolt end distance on beam web angle = 1.25 in. > 1.5 x bolt dia. = 1.5 x .75 =1.125 in. OK

Angle shear

Total shear = 5.986 x 10 bolts = 59.86 kips
Net area = 2 x t (angle length – no. rows x (bolt dia +1/16))
= 2 x .625 x (14.5 – 5 x (3/4 + 1/16))
= 13.05 in.^{2}
Gross area = 2 x t x angle length = 2 x .625 x 14.5
= 18.125 in.2
Net shear = 59.86 / 13.05
= 4.59 ksi < $.3F_u$ = .3 x 58 = 17.4 ksi OK
Gross shear = 59.86 / 18.125
= 3.30 ksi < $.4F_y$ = .4 x 36 = 14.4 ksi OK

Welds angles to beam web

Design is by vector analysis
Horizontal length of weld = angle leg size – 1/2 in. gap = $3^1/_2 - ^1/_2 = 3$ in.
Vertical length of weld = angle length = 14.5 in.
Total weld length = 3 + 14.5+ 3 = 20.5 in.
Polar moment of inertia of weld profile = $(2 \times 3 + 14.5)^3 / 12 - (3^2 (3 + 14.5)^2 / (2 \times 3 + 14.5)$
= 717.92 – 134.45 = 583.47 in^3
Centroid of weld from vertical leg = 3^2 / (2 x 3 + 14.5) = 0.439 in.

Moment on weld = V_{bx} x (angle leg size – centroid distance) = 59.86 x (3½ -..439)
= 183.23 in.-kips

Tensile force at edge of plate = V_{bx} / total length + moment x (horizontal weld length – centroid distance) / I_p
fn = 59.86 / 20.5 + 183.23x (3 – .439) / 583.47 = 2.92 + 0.804
= 3.72 kips / in.

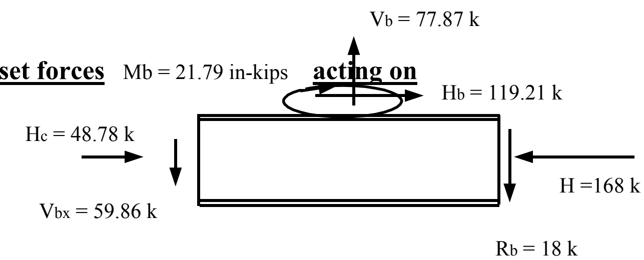
Shear force at edge of plate = H_c / total weld length + moment x (vertical length / 2) / I_p
fv = 48.78 / 20.5 + 183.23 x (14.5/2)/ 583.47
= 2.38 + 2.28 = 4.66 kips / in.

Required fillet weld size = $\sqrt{(3.72)^2 + (4.66)^2} / (0.928 \times 2)$ = 5.96/ (0.928 x 2)
= 3.21 16ths (**use ¼ in**. minimum size for 5/8 material)

Shear stress in beam web @ toe of angles = V_{bcx} / total length – (moment x centroid distance) / I_p
fv′ = 48.78 /20.5 - (183.23 x .439) / 583.47
= 2.38 - 0.14 = 2.24 Kips / in.

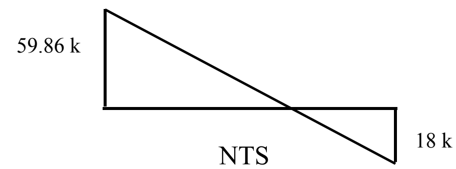
Minimum required beam web thickness
fn / .6Fy = 3.72 / .6 x 50 = 0.124 in. (t_w actual is 0.39 in.) OK
fv / .4Fy = 4.66 / .4 x 50 = 0.233 in. OK
fv′ / .4Fy = 2.24 / .4 x 50 = 0.112 in. OK

Investigate localized beam web shear considering gusset forces acting on beam



Consider a Free Body Diagram of the beam
and the gusset plate and column forces acting on it

The beam shear diagram for these loads is



The maximum local beam web shear is 59.86 kips as shown

Since this is a localized condition use a modified beam shear capacity including the flanges out to k_1 from beam centerline.

Modified shear capacity = (area of beam- 2 (b_f - 2k_1) x t_f) x .4F_y
= (16.2- 2 (7.53 - 2 x .8125)x 0.63) x .4 x 50
= (16.2 – 7.44) x 20
= 175.19 kips >> 59.86 kips OK

Note that for the brace in compression load condition the FBD and shear diagram become;