Heavy Brace Gusset Design Example

Author: *Kevin Cowie; Alistair Fussell*
Affiliation: *Steel Construction New Zealand Inc.*
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Heavy Brace Connection, Gusset, Design Example, Eccentrically Braced Frame, Modified Uniform Force Method.

Introduction
This worked example illustrates the ductile design of brace/beam/column gusset plate connection. The example is based on the design approach presented in Steel Advisor CON1301.

The configuration and details of the gusset plate connection are shown in figure 1.

The brace load N^* has been calculated separately using a capacity design approach.

$N^* = 2000$ kN

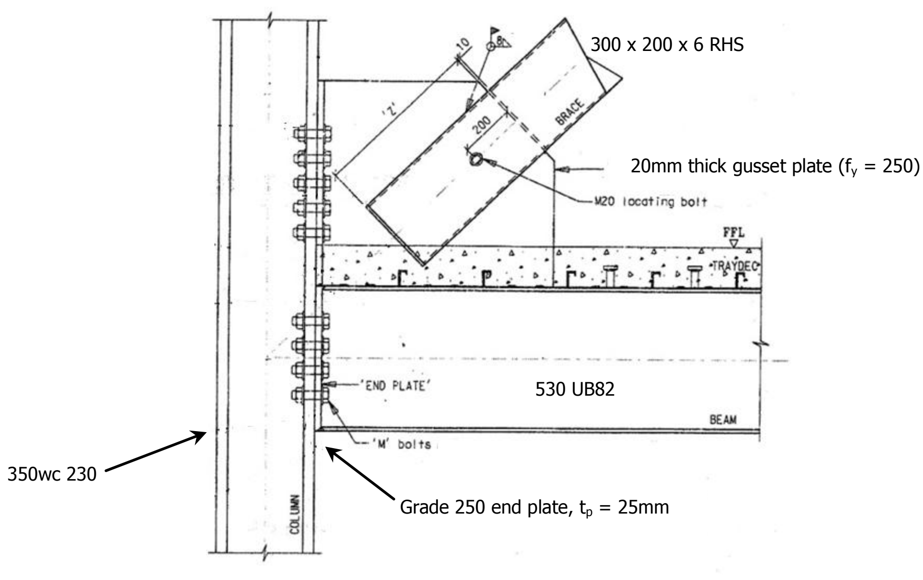
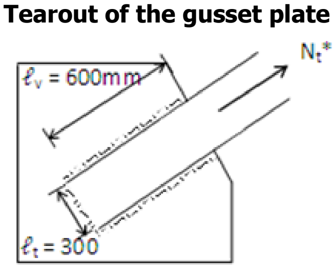


Figure 1: EBF Connection Detail

A) Design of Gusset

1. Tearout of the gusset plate



$N_t^* = 2000$

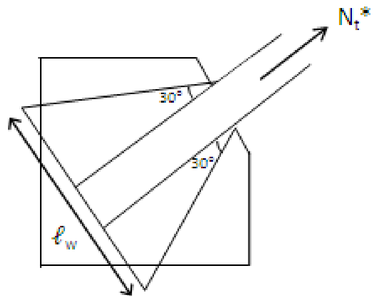
Block Shear Gusset Plate

t_g = gusset plate thickness = 20mm
A_{nt} = 300 x 20 = 6000mm^2
A_{nv} = 600 x 2 x 20 = 24000mm^2

$$\phi N_{BS} = \begin{bmatrix} \phi\,(0.6f_y\,A_{gv} + f_u A_{nt}) \\ \phi\,(0.6f_u\,A_{nv} + f_y\,A_{gt}) \end{bmatrix}_{min}$$
$$= 0.9 \times \min \begin{bmatrix} 0.6 \times 250 \times 24000 + 410 \times 6000 \\ 0.6 \times 410 \times 24000 + 250 \times 6000 \end{bmatrix}$$
$$= 0.9 \times \min\,[6060, 7404]$$
$$= 5454$$

Note: $A_{gt} = A_{nt}$ and $A_{nv} = A_{gt}$ for welded connections.

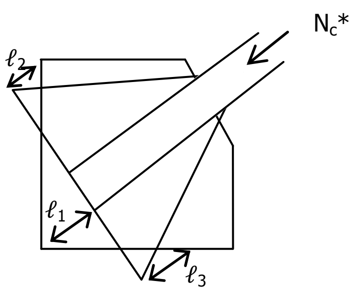
2. Tension Yielding on Whitmore Section of Gusset



ℓ_w = Whitmore section length = 300 + 2 x 600 x tan 30°
= 993mm

The Whitmore section length extends 246mm into the beam web and 86mm into the column web. The web thickness is used, conservatively ignoring the larger flange area.
Web thickness of column = 25mm
Web thickness of beam = 9.6mm
$\phi R_s = \phi A_w f_y$
= 0.9 (661 x 20 x 250 + 86 x 25 x 300 + 246 x 9.6 x 320)
= 4235 kN > N_t^* ∴OK

3. Buckling of the Gusset Plate



The buckling capacity is calculated by the strip of plate defined by the Whitmore section as a column.

The design, L, is taken as average length of ℓ_1, ℓ_2 and ℓ_3. Note for this particular example ℓ_1 and ℓ_3 are negative.

$$L = \frac{\ell_1, \ell_2, \ell_3}{3}$$
$$= \frac{320 + (0) + (0)}{3} \text{ (ignoring negative values)}$$
$$= 107 \text{ mm}$$

The effective length factor, k_e, is 0.7 and is taken from NZS 3404: 1997 cl 12.9.7.2 (b).

The radius of gyration is r = $\frac{t_g}{\sqrt{12}} = \frac{20}{\sqrt{12}}$ = 5.77

The form factor, k_f, is taken as 1.0 and the member section constant, α_b = 0.5

$N_s = R_s$ as determined above.

$\Rightarrow \quad \phi N_c = \phi \alpha_c R_S$

$\lambda_n = \left(\frac{L_e}{r}\right)\sqrt{k_f}\sqrt{\frac{f_y}{250}}$

$= \frac{107}{5.77}\sqrt{1.0}\sqrt{\frac{250}{250}} = 18.5$

From table 6.3.3(2) NZS 3404 α_c= 0.97

$\Rightarrow \quad \phi N_c$ = 0.97 x 4235 = 4108kN > N_c^* $\therefore$OK

4. Check Gusset Edge Stability

$b_g \le \frac{c_1 \, t_p}{\sqrt{f_y/250}}$ (Eqn 10.16 from (Clifton, 1994))

c_1 = 45 for gusset welded to supporting member(s)

$b_g \le \frac{45 \, x \, 20}{\sqrt{250/250}}$

$\le$ 900mm OK!

Maximum unsupported edge length b_g = 540 < 900 OK

5. Determine Gusset Interface Actions

Use modified Uniform Force Method (see paper (Muir, 2008))

From the members and frame geometry

$e_b = \frac{d_b}{2}$ = 264mm

$e_c = \frac{d_c}{2} + t_{ep}$

$= \frac{339}{2}$ + 25 = 194.5mm

Brace angle from vertical

θ = 45°

ℓ_{gb} = length of gusset along beam

= 840mm

t_{ep} = thickness of end plate

= 25mm

$\alpha = \frac{\ell_{gb}}{2} + t_{ep}$

= 445mm

ℓ_{ge} = length of gusset along column

= 740mm

$\beta = \frac{\ell_{gc}}{2}$

= 370mm

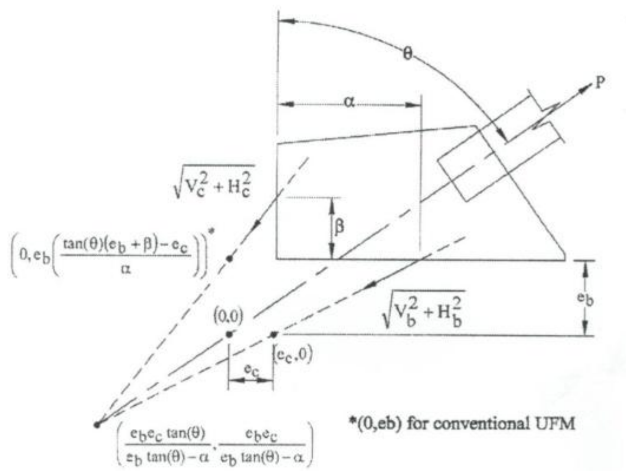


Figure 2: Gusset free body diagram

On the gusset-to-column connection

$H_c^* = \frac{\cos(\theta)e_c}{(e_b+\beta)} \times P^*$

$= 433kN$

$V_c^* = P^* \cos\theta - V_b^*$

$= 832.6kN$

On the gusset-to-beam connections

$V_b^* = \left[\frac{e_b[\sin(\emptyset)(e_b+\beta) - \cos(\theta)e_c]}{\propto(e_b+\beta)}\right] P^*$

$= 581.6kN$

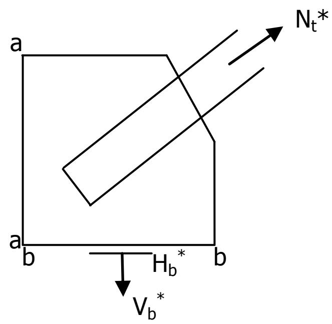
$H_b^* = P^* \sin(\theta) - H_c^*$

$= 981kN$

6. Combine Action Plate Check

The vertical location occurs at the gusset interfaces.

B-B is critical since largest combined actions.



Using modified Uniform Force Method

$H_b^* = 981kN$

$V_b^* = 582kN$

Gusset Local Shear Capacity

$\phi V_g = \phi 0.6\ A_g f_{yg}$

$= 0.9 \times 0.6 \times 840 \times 20 \times 250/1000$

$= 2268kN > H_b^*$ OK

Gusset Local Tensile Capacity

$\phi N_g = \phi\ A_g f_{yg}$

$= 0.9 \times 840 \times 20 \times 250/1000$

$= 3780kN > V_b^*$ OK

Check Combined Stresses at b-b interface

$$\left(\frac{H_b^*}{\phi V_g}\right)^2 + \left(\frac{V_b^*}{\phi N_g}\right)^2 \leq 1.0$$ (Eqn 10.2 from (Clifton, 1994))

$$\left(\frac{981}{2268}\right)^2 + \left(\frac{582}{3780}\right)^2 = 0.21 \leq 1.0$$ OK

7. Gusset Weld to Beam

Distribution of gusset interface stress not uniform, weld designed to develop the design tension capacity at the connection.

For 20mm plate tensile design capacity.

$\phi N_t = \phi f_y A_s$

$= 0.9 \times \frac{250}{1000} \times 20$ / mm

$= 4.5kN/mm$

For 2 parallel welds either side of plate.
$V_w^* = 4.5/2 = 2.25$kN/mm
$\phi V_w = 1.96$kN/mm for 12mm fillet E48 electrode.

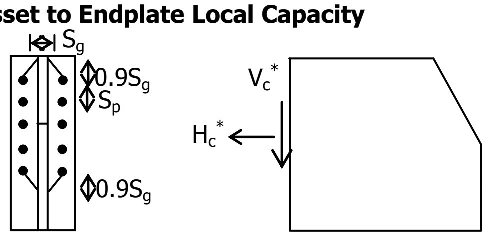
Since $\phi V_w = 1.96 < 2.25$ use full strength butt as generally more cost effective than increasing fillet weld above 12mm.

8. Gusset Weld to End Plate

Distribution of forces not uniform, weld designed to develop the design tension capacity at the connection. Calculations as per (7) above, use full strength butt weld.

B) End Plate

1. Gusset to Endplate Local Capacity



Using modified Uniform Force Method
$H_c^* = 433$kN
$V_c^* = 832$kN

Need to consider gusset length that is effective in transferring through endplate into bolts.
$0.9S_g = 0.9 \times 140 = 126$mm
$S_p = 90$mm
$\ell_{ge} = 2 \times 126 + 90 \times 4 = 612$mm

Gusset local shear capacity
$\phi V_g = \phi 0.6\, f_{yg}\, \ell_{ge} t_g$
$= 0.9 \times 0.6 \times 612 \times 20 \times 250/1000$
$= 1652\text{kN} > V_c^*$ OK

Gusset local tension capacity
$\phi N_g = \phi\, \ell_{ge} t_g\, f_{yg}$
$= 0.9 \times 612 \times 20 \times 250/1000$
$= 2754\text{kN} > H_c^*$ OK

Check Combined

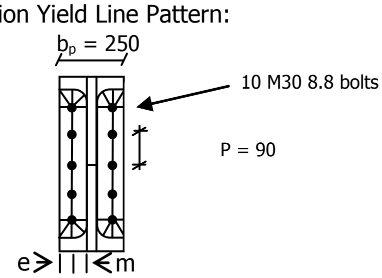
$$\left(\frac{H_c^*}{\phi N_g}\right)^2 + \left(\frac{V_c^*}{\phi V_g}\right)^2 \le 1.0$$

$$\left(\frac{433}{2754}\right)^2 + \left(\frac{832}{1652}\right)^2 = 0.28 \le 1.0 \quad \text{OK}$$

2. Check Tension Capacity of Endplate

The yield line solutions from SCI publication p207/95 "Joints in Steel Construction – Moment Connections" have been used to determine the tensile capacity of the gusset end plate.

Tension Yield Line Pattern:



Equivalent effective T-stub length for group. (pp35 SCI 207/95)

$m = \frac{(140-20)}{2} = 60\text{mm}$

$\ell_e = 4m + 1.25e + 4p$
$= 3 \times 60 + 1.25 \times 55 + 4 \times 90$
$= 609\text{mm}$

Mode 1: 4 Plastic Hinges in T-Stub (pp44 HERA Report R4-142)

$\phi N_1 = \frac{\emptyset_s f_{yi} \ell_e t_i^2}{m}$
$= \frac{0.9 \times 250\ MPa \times 609 \times 25^2}{60}$
$= 1{,}427\text{ kN}$

Mode 2: 2 Plastic Hinges in T-Stub (pp45 HERA Report R4-142)

$n = \min\left[\frac{b_p}{2} - \frac{S_g}{2}; 1.25 m_1\right]$
$= \min\left[\frac{250-140}{2}; 1.25 \times 60\right]$
$= \min[55, 75] = 55\text{mm}$

$\phi N_2 = \frac{0.5\,\emptyset_s f_{yi} \ell_e t_i^2 + n\,10\,\emptyset_b N_{tf}}{m+n}$
$= \frac{0.5 \times 0.9 \times 250 \times 609 \times 25^2 + 55 \times 10 \times 373}{60+55}(10^3)$
$= 2156\text{ kN}$

Mode 3: Bolt only mode

$\phi N_3 = 10\,\phi_b N_{tf}$
$= 10 \times 373$
$= 3730\text{ kN}$

$\Rightarrow$ Mode 1 Governs > H_c^* $\therefore$ OK! (This will ensure ductile behaviour as brittle mode 3 suppressed by strength hierarchy)

3. Brace Gusset/Column Bolted Connection

Try 6 M30 bolts.

$V_c = 833\text{kN}$
$H_c = 433\text{kN}$ (determined using modified uniform force method)

Endplate thickness = 25mm

Check complies with maximum plate thickness.
$T_{p,max} \leq 0.9\, d_f = 0.9 \times 30 = 27\text{mm} > 25$ OK
Allow 40% extra for prying action

$T^* = \frac{433 \times 1.4}{6} = 101\text{kN}$
$V^* = \frac{833}{6} = 139\text{kN}$
$\phi V_f = 214\text{kN}$ M30 threads included
$\phi N_f = 373\text{kN}$

Combined check

$$\left(\frac{V^*}{\phi V_f}\right)^2 + \left(\frac{N^*}{\phi N_{tf}}\right)^2 = \left(\frac{101}{372}\right)^2 + \left(\frac{139}{214}\right)^2 = 0.50 < 1.0 \quad \text{OK}$$

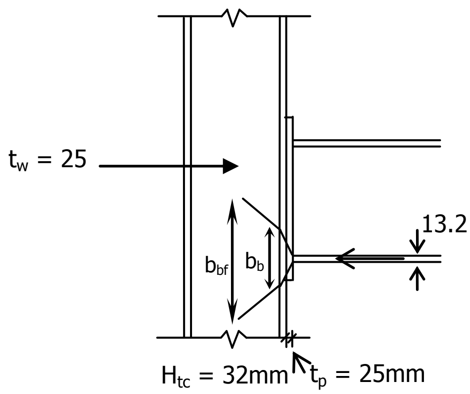
C. COLUMN FLANGE TENSION CHECK

The column flange is wider and thicker than the end plate (ie H_{tc} = 32 > 25mm). The column web is thicker than the gusset plate. And so, by inspection, the column flange will be stronger than the end plate. Therefore, the appropriate strength hierarchy has been achieved.

D. COLUMN WEB CRIPPLING CHECK

Check if column can sustain compression loading from bottom flange if collector beam section capacity ϕM_s is developed due to frame action induced moment.

N^* = $\phi f_y A_f$ = 0.9 x 320 x 2758 = 794kN
f_y = 320 MP_a
A_f = 209 x 13.2 = 2758mm^2



Spread of load – 1:2.5 through flanges, 1:1 through web.

Bearing width
b_b = 13.2 + 2.5 x 2 (32+25) + 2 x $\left(\frac{339}{2} - 32\right)$
= 575mm
b_{bf} = 13.2 + 2.5 x 2 (32+25) = 298.2mm

1. Bearing Yielding Capacity

ϕR_{by} = ϕ 1.25 $b_{bf} t_w f_y$
= 0.9 x 1.25 x 298 x 25 x 300 = 2514kN > 794 OK

2. Bearing Buckling Capacity

ϕR_{bb} = $\phi \alpha_c A_{bb} f_y$
A_{bb} = b_{bb} A_w = 575 x 25
= 14375mm^2
$\frac{Le}{r}$ = $2.5\frac{d_1}{t_w}$ d_1 = 339 – 32 x 2 = 275mm
$\frac{Le}{r}$ = $2.5\,\frac{275}{25}$ = 27.5

for α_b = 0.5 k_f = 1.0
α_c = 0.917 for $\frac{Le}{r}$ = 30 table 6.3.3 (2)

ϕR_{bb} = 0.9 x 14375 x 0.917 x 280
= 3322kN > 793kN OK

No compression stiffener required.

Note: Beam to connection calculations has not been under taken. The design checks for this connection are similar to these in section B.

A check is also required of the collector beam web for shear and tension.

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