6 BRACING CLEAT

6.1 GENERAL

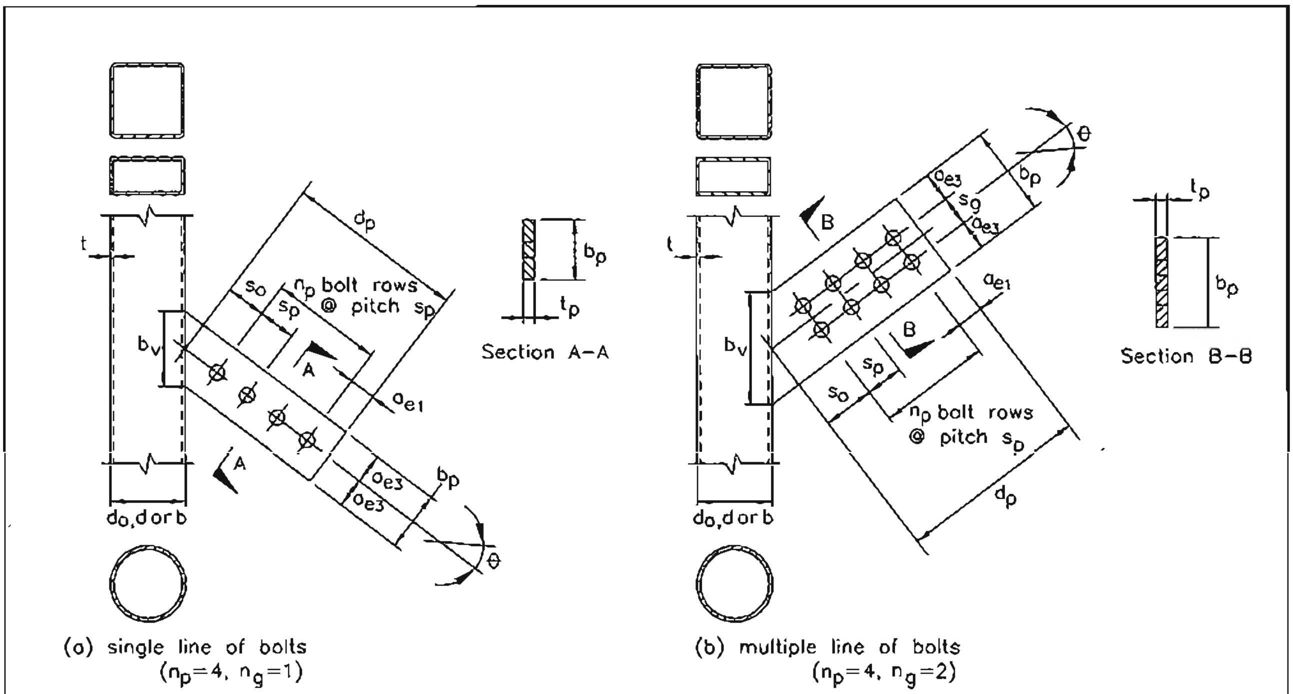

Fig. 6.1-1: Definition of Bracing Cleat Symbols

d_p = depth of bracing cleat along longitudinal centreline
b_p = width of bracing cleat
t_p = thickness of bracing cleat
b_v = projection of bracing cleat on to column face
$= \dfrac{b_p}{\cos\theta}$
θ = angle of inclination of centreline of bracing cleat to horizontal
d = depth of hollow section for RHS/SHS
$= d_o$ for CHS
d_o = outside diameter of CHS
b = width of hollow section for RHS/SHS
$= d_o$ for CHS
t = thickness of hollow section
s_p = pitch of bolts
$\geq 2.5\, d_f$ (Clause 9.6.1 of AS 4100)
s_g = gauge of bolts
$\geq 2.5\, d_f$ (Clause 9.6.1 of AS 4100)
s_o = minimum distance along longitudinal centreline of bracing cleat from face of section to centreline of nearest row of bolts
a_{e1}, a_{e3} = minimum distance from nearer edge of hole to physical edge of bracing cleat plus half bolt diameter
$\geq 1.5\, d_f$ (Table 9.6.2 of AS 4100)
d_f = diameter of bolt

The bracing cleat dimensions (d_p, b_p) are determined from the equations below. See Section 2.2 for definitions and suggested detailing dimensions.

$$d_p = a_{e1} + s_p (n_p - 1) + s_o$$

$$b_p = 2\, a_{e3} + s_g (n_g - 1)$$

where n_p = number of rows of bolts
n_g = number of lines of bolts

Design Action Effects:

Axial Tension	Section 6.2
Axial Compression	Section 6.3

References:

[6.1] Hogan, T.J., Thomas, I.R., "Design of Structural Connections", 4th ed., Australian Institute of Steel Construction, 1994.

[6.2] Packer, J.A., Henderson, J.E., "Design Guide for Hollow Structural Section Connections", Canadian Institute of Steel Construction, 1992.

[6.3] Packer, J.A., Wardenier, J., Kurobane, Y., Dutta, D., and Yeomans, N., "Design Guide for Rectangular Hollow Section (RHS) Joints under Predominantly Static Loading", CIDECT (Ed.) and Verlag TÜV Rheinland GmbH, Köln, Federal Republic of Germany, 1992.

[6.4] Wardenier, J., Kurobane, Y., Packer, J.A., Dutta, D., and Yeomans, N., "Design Guide for Circular Hollow Section (CHS) Joints under Predominantly Static Loading", CIDECT (Ed.) and Verlag TÜV Rheinland GmbH, Köln, Federal Republic of Germany, 1991.

[6.5] Kitipornchai, S., Al-Bermani, F.G.A., Murray, N.R., "Eccentrically Connected Cleat Plates in Compression", *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 119, No. 3, 1993, pp 767-781.

[6.6] Packer, J.A., "Design with Structural Steel Hollow Sections", *Australian Institute of Steel Construction Seminar Proceedings*, Australian Institute of Steel Construction, 1996.

Notes: 1. The design model considers the following:
   (a) Simple isolated bracing connections with one bracing member, which is a common connection configuration (Ref[6.1]). Most studies of bracing connections have considered two or more bracing members. Reference should be made to Section 5.11 of Ref[6.1] and Sections 6.2.1 and 6.3.1 of this Part for guidance on design of SSHS column bracing connections with two or more members.
   (b) Bracing cleat configurations are as shown in Fig. 6.1-1 (i.e. the bracing cleat longitudinal edges are parallel to the longitudinal axis of the brace member). Reference should be made to Section 5.11 of Ref[6.1] for guidance on the design of rectangular bracing cleats.

2. The design model is based on the following design assumptions:
   (a) Other than noted in (e)(iii) below, there are generally no eccentricity effects considered.
   (b) The cleat is only loaded axially.
   (c) The weld is only designed to transmit shear force transverse to the throat and shear force longitudinal to the throat, with no bending moment.
   (d) The bolts are loaded in shear and are assumed to be uniformly loaded unless the line of bolts is very long (see Section 2.3.4.2).
   (e) For the design of the bracing cleat connected to *SSHS bracing members* –
      (i) the centreline of the bolt group, attached cleat component and fillet weld group are assumed to be collinear (Fig. 6.1-2(a)).
      (ii) except as noted in (e)(iii) below, the design of the bracing cleat and SSHS column for axial tension and axial compression loadings is to be in accordance with Part 6.
      (iii) the component buckling design of the bracing cleat as noted in Section 6.3.2.3 is not considered valid. The appropriate failure mode should take account of the flexibilty of the end connection in SSHS bracing members (e.g. flattened end, welded tee end and slotted end plate) in which the connection eccentricity for axial compression loadings in Fig. 6.1-2(c) is used. Consequently, cleat component buckling design should be in accordance with Sections 7.3.1.3, 8.3.2.3 and 9.3.1.3 as appropriate for the respective SSHS end connection geometry.
   (f) For the design of the bracing cleat connected to *open section bracing members* (e.g. angles, channels, I-sections) –
      (i) for angles, the gravity axis of the attached bracing member and the centreline of the connection are not assumed to be collinear (Fig. 6.1-2(b)). However, as long as conventional gauge lines are used for the attached bracing member (see Section 2.2) to establish the centreline of the connection, this eccentricity has been shown in tests to have negligible effect on the static strength of members.
      (ii) the design of the SSHS column and bracing cleat for axial tension and axial compression brace loadings is to be in accordance with the appropriate sections of Part 6. This includes the component buckling design of the bracing cleat, which due to the stiffness of the open section connected to the cleat, assumes the non-sway buckling mode as noted in (f)(iii) below.
      (iii) the design of the bracing cleat eccentrically connected in axial compression assumes that the cleat is a short column fixed at both ends (Ref[6.1]). This suggests a "member" effective length factor of 0.7 from Table 4.6.3.2 of AS 4100. Ref[6.5] should be consulted for a more rigorous method to determine the design capacity of eccentrically connected cleat components in axial compression.

   Based on the above eccentricity assumptions, the design model should only be used for statically loaded brace members.

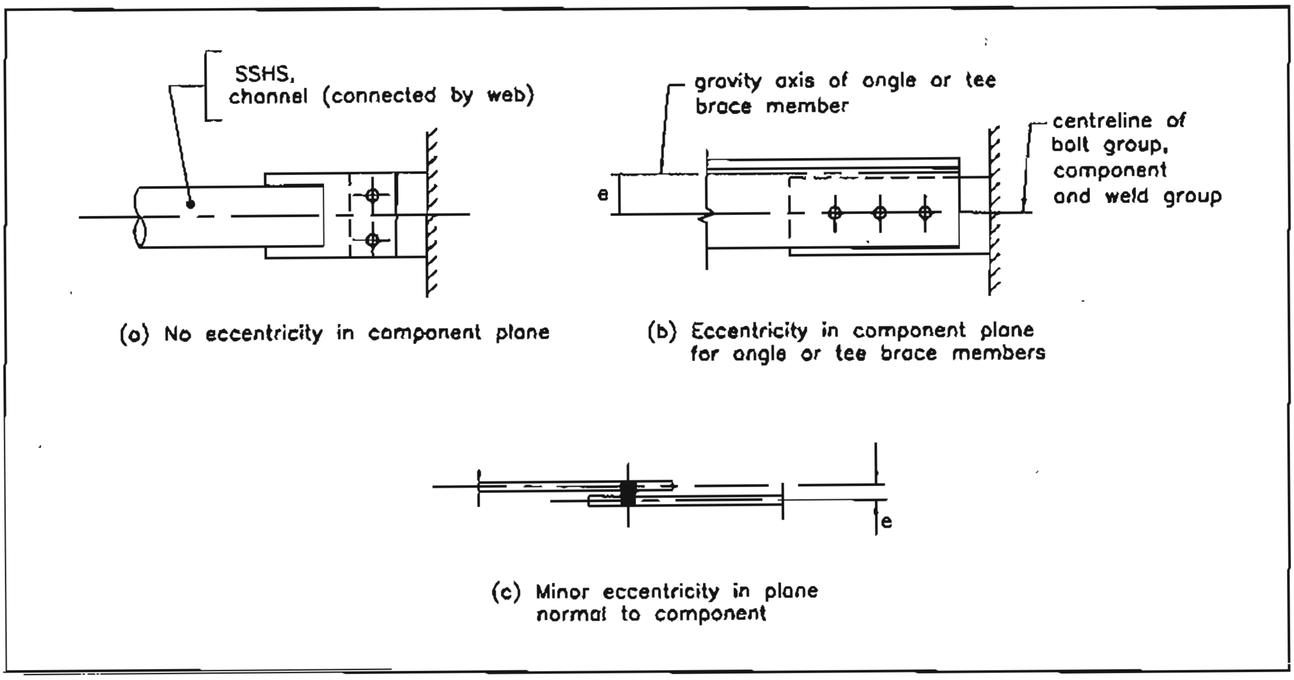

Fig. 6.1-2: Eccentricity in Bracing Cleat

3. Cleat components should be detailed as rectangular shapes to reduce marking-off and cutting time.
4. Wherever possible square edge flat bars should be used as connection components.
5. For economy, either 4.6/S or 8.8/S bolting category is preferred. Category 8.8/S is commonly used.
6. Clause 9.1.4 of AS 4100 requires connections at the ends of compression or tension members to be designed for a minimum force of 0.3 times the member design capacity.

$$N_{des} \geq 0.3\, \phi N_t \text{ or } 0.3\, \phi N_c$$

6.2 AXIAL TENSION

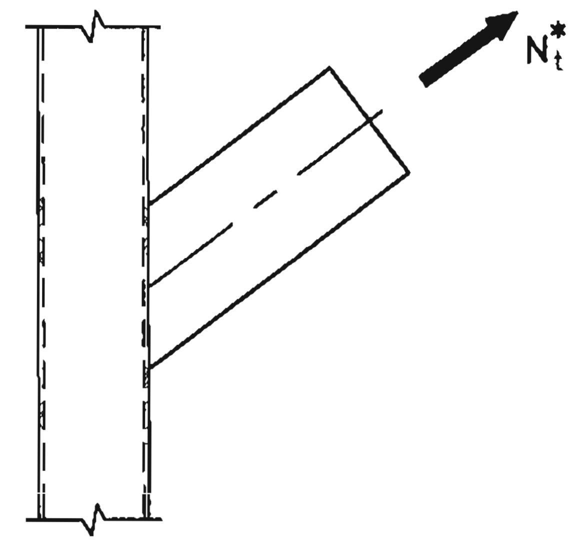

Fig. 6.2: Bracing Cleat subject to Axial Tension

The connection design capacity (N_{des}) of the bracing cleat subject to axial tension is the least value obtained from the strength limit state equations in Sections 6.2.1 to 6.2.4. For design, the requirement is the connection capacity must be greater than or equal to the design axial tension force (N_t^*).

$$N_{des} = \min[N_a, N_b, N_c, N_d, N_e, N_f, N_g, N_h, N_i] \geq N_t^*$$

where N_a, N_b, N_c, N_d, N_e, N_f, N_g, N_h, N_i are defined below.

6.2.1 Hollow Section Design

The design capacities (N_a, N_b) for the hollow section are based on the design capacities in axial tension for shearing of the SSHS column wall and yielding of the SSHS column face.

6.2.1.1 Wall Shear

The design capacity (N_a) based on the design shear capacity (ϕN_V) of the SSHS column wall is:

$$N_a = \frac{2\,\phi N_V}{\sin\theta}$$

where

$\phi N_V = \phi\, 0.6\, f_y\, A_w$ . . . . Clause 5.11.4 of AS 4100

where ϕ = 0.9 (Table 3.4 of AS 4100)
f_y = yield stress of SSHS column
$A_w = b_v t$

6.2.1.2 Column Face Yielding

The design capacity (N_b) based on the design section capacity in axial tension (ϕN_H) of the hollow section for column face yielding is:

$$N_b = \frac{\phi N_H}{\cos\theta}$$

where for RHS and SHS column

$$\phi N_H = \frac{f_y t^2}{(1-\beta)}\left(2\eta + 4\sqrt{1-\beta}\right) \quad \text{.......... Refs[6.2, 6.3]}$$

which is valid for

$$\left(\frac{b}{t}\right) \leq 30$$

where f_y = yield stress of RHS/SHS column

$$\beta = \frac{t_p}{b}$$

$$\eta = \frac{b_v}{b}$$

Note: b is the width of RHS/SHS column face with bracing cleat attached. When the bracing cleat is attached to the long side of the RHS, the dimensions b and d are interchanged in the ratios b/t, β and η.

and for CHS column

$$\phi N_H = 5.0 f_y t^2 (1 + 0.25\eta) f(n') \quad \text{.......... Refs[6.2, 6.4]}$$

which is valid for

$$\eta \leq 4$$

where f_y = yield stress of CHS column

$$\eta = \frac{b_v}{d_o}$$

$f(n')$ = 1.0 (CHS column unloaded or in axial tension)

$$= 1 + 0.3n' - 0.3n'^2 \quad \text{for } n' < 0 \quad \text{(CHS column in axial compression)}$$

$$n' = -\left(\frac{N^*_{0p}}{N'_s} + \frac{M^*_0}{M'_s}\right)$$

N^*_{0p} = design preload (i.e. additional axial compression in CHS column at connection other than that required to maintain equilibrium with bracing cleat expressed as an absolute value)
N'_s = $f_y A_g$ (for CHS column)
M^*_0 = design bending moment in CHS column
M'_s = $f_y Z_0$ (for CHS column)
A_g = gross area of cross-section of CHS column
Z_0 = elastic section modulus of CHS column

6.2.2 Component Design

The design capacities (N_c, N_d, N_e, N_f, N_g) for the cleat component are based on the design capacities in axial tension for yield, fracture, bearing, tearout and block shear.

6.2.2.1 Yield

The design capacity (N_c) based on the design section capacity in axial tension (ϕN_t) of the cleat component for yielding is:

$$N_c = \phi N_t$$

where

$\phi N_t = \phi A_g f_{yp}$ . . . . see Section 2.3.2.1

6.2.2.2 Fracture

The design capacity (N_d) based on the design section capacity in axial tension (ϕN_t) of the cleat component for fracture is:

$$N_d = \phi N_t$$

where

$\phi N_t = \phi\, 0.85\, k_t f_{up} A_n$ . . . . see Section 2.3.2.1

6.2.2.3 Bearing

The design capacity (N_e) based on the design bearing capacity (ϕV_b) of the cleat component due to a bolt in shear is:

$$N_e = n_b \phi V_b$$

where

$\phi V_b = \phi\, 3.2\, d_f t_p f_{up}$ . . . . see Section 2.3.2.2

where n_b = number of bolts in bolt group

6.2.2.4 Tearout

The design capacity (N_f) based on the design bearing capacity (ϕV_b) of the cleat component due to tearout of a bolt in shear is:

$$N_f = n_b \phi V_b$$

where

$\phi V_b = \phi\, a_{ex}\, t_p\, f_{up}$ .... see Section 2.3.2.2

where n_b = number of bolts in bolt group

6.2.2.5 Block Shear

The design capacity (N_g) based on the design capacity in axial tension (ϕN_{tb}) of the cleat component for block shear is:

$$N_g = \phi N_{tb}$$

where ϕN_{tb} is the *greater* of

$\phi N_{tb} = \phi\,(0.6\, f_{yp}\, A_{vg} + f_{up}\, A_{nt})$ .... see Section 2.3.2.3

and

$\phi N_{tb} = \phi\,(0.6\, f_{up}\, A_{ns} + f_{yp}\, A_{tg})$ .... see Section 2.3.2.3

6.2.3 Weld Design

The design capacity (N_h) based on the design capacity (ϕv_w) of the fillet weld per unit length is:

$$N_h = 2\, \phi v_w\, L_w$$

where

$\phi v_w = \phi\, 0.6\, f_{uw}\, t_t\, k_r$ .... see Section 2.3.3

where ϕ = capacity factor (see Section 2.3.3.1)
L_w = length of fillet weld
= b_v

6.2.4 Bolt Design

The design capacity (N_i) based on the design shear capacity (ϕV_{df}) of the bolt group is:

$$N_i = \phi V_{df}$$

where

$$\phi V_{df} = \phi \, n_b \, V_f$$

where ϕ = 0.8 (Table 3.4 of AS 4100)
n_b = number of bolts in bolt group
V_f = nominal shear capacity of single bolt
= $0.62 \, f_{uf} \, k_r \, A_v$ (see Section 2.3.4.2)

6.3 AXIAL COMPRESSION

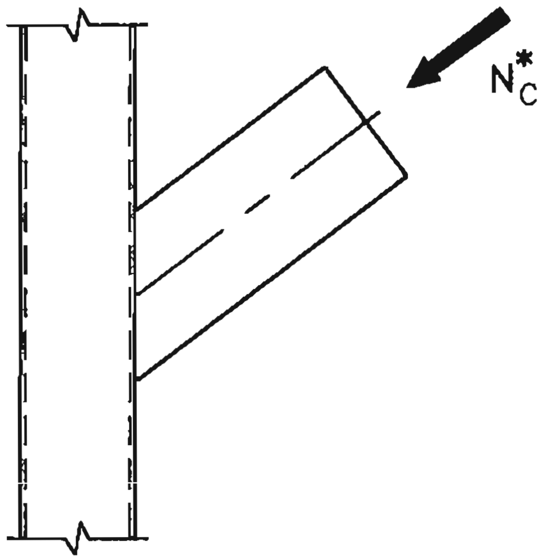

Fig. 6.3: Bracing Cleat subject to Axial Compression

The connection design capacity (N_{des}) of the bracing cleat subject to axial compression is the least value obtained from the strength limit state equations in Sections 6.3.1 to 6.3.4. For design, the requirement is the connection capacity must be greater than or equal to the design axial compression force (N_c^*).

$$N_{des} = \min[N_a, N_b, N_c, N_d, N_e, N_f, N_g] \geq N_c^*$$

where N_a, N_b, N_c, N_d, N_e, N_f, N_g are defined below.

6.3.1 Hollow Section Design

The design capacities (N_a, N_b) for the hollow section are based on the design capacities in axial compression for shearing of the SSHS column wall and yielding of the SSHS column face.

6.3.1.1 Wall Shear

The design capacity (N_a) based on the design shear capacity (ϕN_V) of the SSHS column wall is:

$$N_a = \frac{2\,\phi N_V}{\sin\theta}$$

where

$$\phi N_V = \phi\, 0.6\, f_y\, A_w$$ .... Clause 5.11.4 of AS 4100

where ϕ = 0.9 (Table 3.4 of AS 4100)
f_y = yield stress of SSHS column
A_w = $b_v\, t$

6.3.1.2 Column Face Yielding

The design capacity (N_b) based on the design section capacity in axial compression (ϕN_H) of the hollow section for column face yielding is:

$$N_b = \frac{\phi N_H}{\cos\theta}$$

where for RHS and SHS column

$$\phi N_H = \frac{f_y t^2}{(1-\beta)}\left(2\eta + 4\sqrt{1-\beta}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{Refs[6.2, 6.3]}$$

which is valid for

$$\left(\frac{b}{t}\right) \leq 30$$

where f_y = yield stress of RHS/SHS column

$$\beta = \frac{t_p}{b}$$

$$\eta = \frac{b_v}{b}$$

Note: b is the width of RHS/SHS column face with the bracing cleat attached. When the bracing cleat is attached to the long side of the RHS, the dimensions b and d are interchanged in the calculation of the ratios b / t, β and η.

and for CHS column

$$\phi N_H = 5.0 f_y t^2 (1 + 0.25\eta)\, f(n') \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{Refs[6.2, 6.4]}$$

which is valid for

$$\eta \leq 4$$

where f_y = yield stress of CHS column

$$\eta = \frac{b_v}{d_o}$$

$f(n')$ = 1.0 (CHS column unloaded or in axial tension)
= $1 + 0.3n' - 0.3n'^2$ for $n' < 0$ (CHS column in axial compression)

$$n' = -\left(\frac{N^*_{0p}}{N'_s} + \frac{M^*_0}{M'_s}\right)$$

N^*_{0p} = design preload (i.e. additional axial compression in CHS column at connection other than that required to maintain equilibrium with bracing cleat expressed as an absolute value)
N'_s = $f_y A_g$ (for CHS column)
M^*_0 = design bending moment in CHS column
M'_s = $f_y Z_0$ (for CHS column)
A_g = gross area of cross-section of CHS column
Z_0 = elastic section modulus of CHS column

6.3.2 Component Design

The design capacities (N_c, N_d, N_e) for the cleat component are based on the design capacities in axial compression for bearing, tearout and buckling.

6.3.2.1 Bearing

The design capacity (N_c) based on the design bearing capacity (ϕV_b) of the cleat component due to a bolt in shear is:

$$N_c = n_b \phi V_b$$

where

$\phi V_b = \phi\, 3.2\, d_f t_p f_{up}$ .... see Section 2.3.2.2

where n_b = number of bolts in bolt group

6.3.2.2 Tearout

The design capacity (N_d) based on the design bearing capacity (ϕV_b) of the cleat component due to tearout of a bolt in shear is:

$$N_d = n_b \phi V_b$$

where

$\phi V_b = \phi\, a_{e2} t_p f_{up}$ ....see Section 2.3.2.2

where n_b = number of bolts in bolt group

6.3.2.3 Buckling

The design capacity check for buckling of the component subject to axial compression only applies to bracing cleats connected to open section brace members (e.g. angle, channel, I-section). For cleats connected to SSHS brace members refer to Note 2 in Section 6.1.

The design capacity (N_e) based on the design member capacity in axial compression (ϕN_c) of the cleat component is:

$$N_e = \phi N_c$$

where

$$\phi N_c = \phi\, \alpha_c\, N_s \leq \phi N_s$$ .... see Clause 6.3.3 of AS 4100

where ϕ = 0.9 (Table 3.4 of AS 4100)

$$\alpha_c = \xi \left\{ 1 - \sqrt{\left[1 - \left(\frac{90}{\xi \lambda} \right)^2 \right]} \right\}$$

$$\xi = \frac{\left(\frac{\lambda}{90} \right)^2 + 1 + \eta}{2 \left(\frac{\lambda}{90} \right)^2}$$

$$\lambda = \lambda_n + \alpha_a\, \alpha_b$$

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

$L_e = 0.7\, s_o$ (see Fig. 6.3.2.3)

$$r = \sqrt{\frac{t_p^2}{12}}$$

k_f = 1.0 (see Section 2.3.2.6)

f_{yp} = yield stress of component

$$\alpha_a = \frac{2100\,(\lambda_n - 13.5)}{\lambda_n^2 - 15.3\,\lambda_n + 2050}$$

α_b = 0.5

$\eta = 0.00326\,(\lambda - 13.5) \geq 0$

$N_s = k_f\, A_n\, f_{yp}$ (see Section 2.3.2.6)

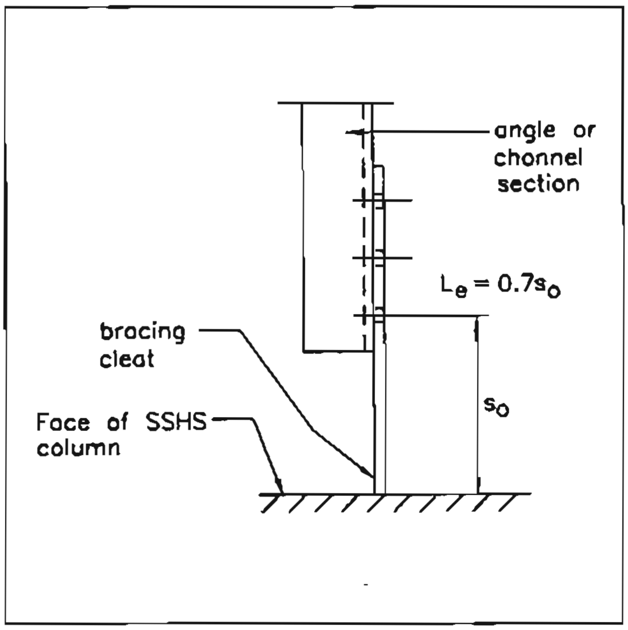

Fig. 6.3.2.3: Bracing Cleat Configuration for Buckling Check

6.3.3 Weld Design

The design capacity (N_f) based on the design capacity (ϕv_w) of the fillet weld per unit length is:

$$N_f = 2\,\phi v_w\,L_w$$

where

$$\phi v_w = \phi\,0.6\,f_{uw}\,t_t\,k_r \quad \text{...... see Section 2.3.3}$$

where ϕ = capacity factor (see Section 2.3.3.1)
L_w = length of fillet weld
= b_v

6.3.4 Bolt Design

The design capacity (N_g) based on the design shear capacity (ϕV_{df}) of the bolt group is:

$$N_g = \phi V_{df}$$

where

$$\phi V_{df} = \phi\,n_b\,V_f$$

where ϕ = 0.8 (Table 3.4 of AS 4100)
n_b = number of bolts in bolt group
V_f = nominal shear capacity of single bolt
= $0.62\,f_{uf}\,k_r\,A_v$ (see Section 2.3.4.2)

6.4 DESIGN EXAMPLE

EXAMPLE 1: BRACING CLEAT IN AXIAL TENSION

Design Parameters

Bracing Cleat for a Grade C350 200x200x9.0 SHS column, as shown in Fig. 6.4.1, subject to the following design action:

Axial Tension $N_t^* = 150$ kN

Note: The design of the welded tee end SSHS bracing member is considered in Example 1 of Section 8.4.

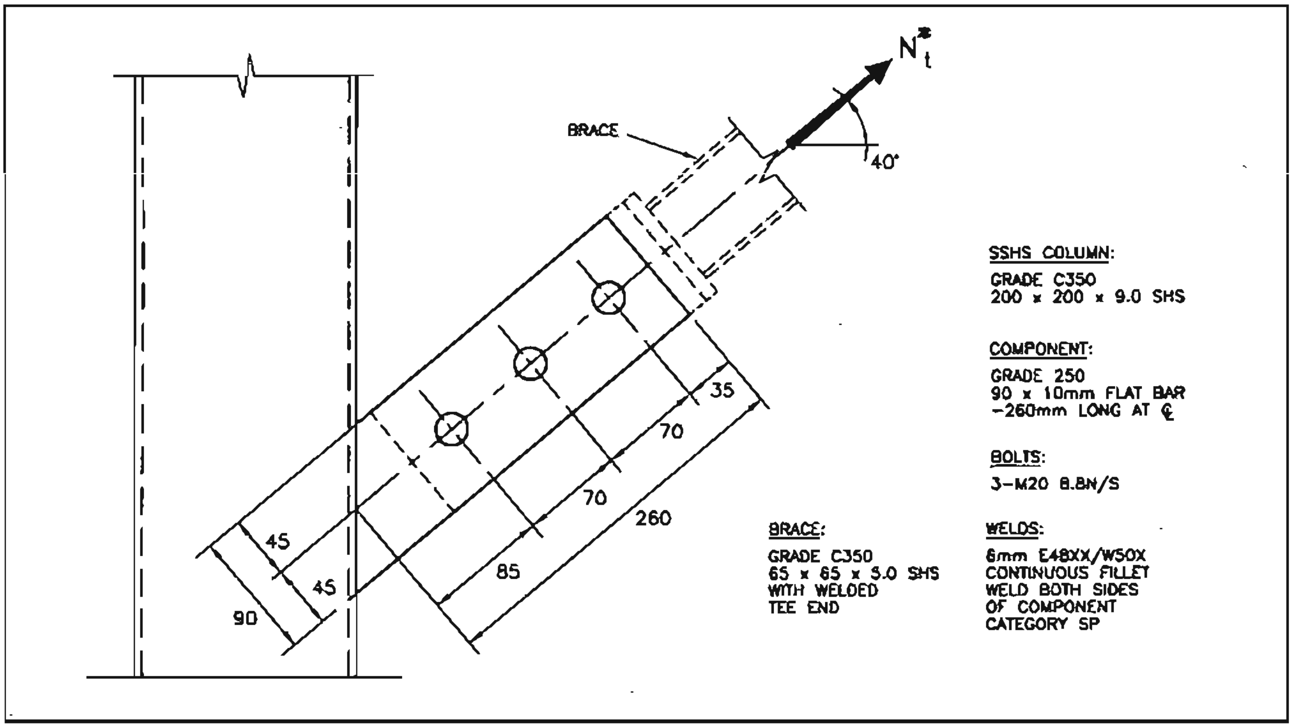

Fig. 6.4.1: Bracing Cleat Geometry

Connection Geometry

$d_p = 260$ mm	$s_p = 70$ mm	$n_p = 3$	$f_y = 350$ MPa
$b_p = 90$ mm	$s_g = 0$ mm	$n_g = 1$	$f_u = 430$ MPa
$t_p = 10$ mm	$s_o = 85$ mm	$n_b = 3$	$f_{yp} = 260$ MPa
$d = 200$ mm	$a_{e1} = 35$ mm	$d_f = 20$ mm	$f_{up} = 410$ MPa
$b = 200$ mm	$a_{e2} = 70 - \dfrac{22}{2} = 59$ mm	$d_h = 22$ mm	$f_{uw} = 480$ MPa
$t = 9.0$ mm		$t_w = 6$ mm	
$\theta = 40°$	$a_{e3} = 45$ mm	$b_v = \dfrac{90}{\cos 40°} = 117$ mm	

Minimum Design Action Effects

For Grade C350 65x65x5.0 SHS brace member:

$$\phi N_t = 351 \text{ kN}$$

$$0.3\,\phi N_t = 0.3 \times 351 = 105 \text{ kN} < N_t^* = 150 \text{ kN}$$ Satisfactory — Ref[2.3.1] Note 6 of 6.1

$\therefore N_t^* = 150$ kN is adequate.

Design Capacity of the Connection

(i) Column wall shear capacity to Section 6.2.1.1

$$N_a = \frac{2 \times 0.9 \times 0.6 \times 350 \times 117 \times 9.0 \times 10^{-3}}{\sin 40°} = 619 \text{ kN}$$ 6.2.1.1

$> N_t^*$ Satisfactory

(ii) Column face yielding capacity to Section 6.2.1.2

Check SSHS column face slenderness validity range:

$$\frac{b}{t} = \frac{200}{9.0} = 22.2$$ 6.2.1.2

< 30 Satisfies validity range

$$\beta = \frac{10}{200} = 0.05$$

$$\eta = \frac{117}{200} = 0.585$$

$$\phi N_H = \frac{350 \times 9.0^2}{(1 - 0.05)} \times \left(2 \times 0.585 + 4 \times \sqrt{1 - 0.05}\right) \times 10^{-3} = 151 \text{ kN}$$

$$N_b = \frac{151}{\cos 40°} = 197 \text{ kN}$$ 6.2.1.2

$> N_t^*$ Satisfactory

(iii) Component yield capacity to Section 6.2.2.1

$$N_c = 0.9 \times 90 \times 10 \times 260 \times 10^{-3} = 211 \text{ kN}$$ 6.2.2.1

$> N_t^*$ Satisfactory

(iv) Component fracture capacity to Section 6.2.2.2

$$A_n = 90 \times 10 - 1 \times 22 \times 10 = 680 \text{ mm}^2$$

$$N_d = 0.9 \times 0.85 \times 1.0 \times 410 \times 680 \times 10^{-3} = 213 \text{ kN}$$ 6.2.2.2

$> N_t^*$ Satisfactory

(v) Component (bolt) bearing capacity to Section 6.2.2.3

N_e = $3 \times 0.9 \times 3.2 \times 20 \times 10 \times 410 \times 10^{-3}$ = 708 kN 6.2.2.3
$> N_t^*$ Satisfactory

(vi) Component (bolt) tearout capacity to Section 6.2.2.4

a_{ex} = min[35, 59] = 35 mm

N_f = $3 \times 0.9 \times 35 \times 10 \times 410 \times 10^{-3}$ = 387 kN 6.2.2.4
$> N_t^*$ Satisfactory

(vii) Component block shear capacity to Section 6.2.2.5

A_{vg} = $10 \times [35 + 70 \times (3 - 1)]$ = 1 750 mm^2
A_{tg} = 45×10 = 450 mm^2
A_{nt} = $450 - (0.5 \times 22 \times 10)$ = 340 mm^2
A_{ns} = $1750 - [22 \times 10 \times (3 - 0.5)]$ = 1 200 mm^2

ϕN_{tb} is the greater of:
ϕN_{tb} = $0.9 \times (0.6 \times 260 \times 1\,750 + 410 \times 340) \times 10^{-3}$ = 371 kN
ϕN_{tb} = $0.9 \times (0.6 \times 410 \times 1\,200 + 260 \times 450) \times 10^{-3}$ = 371 kN

$\therefore N_g$ = 371 kN 6.2.2.5
$> N_t^*$ Satisfactory

(viii) Weld design to Section 6.2.3

ϕv_w = 0.978 kN/mm T2.3.3.2-1

N_h = $2 \times 0.978 \times 117$ = 229 kN 6.2.3
$> N_t^*$ Satisfactory

(ix) Bolt design to Section 6.2.4

ϕV_{fn} = 92.7 kN T2.3.4.4-2

N_i = 3×92.7 = 278 kN 6.2.4
$> N_t^*$ Satisfactory

EXAMPLE 2: BRACING CLEAT IN AXIAL COMPRESSION CONNECTED TO AN ANGLE BRACE MEMBER

Design Parameters

Bracing Cleat for a Grade C350 150x150x6.0 SHS column, as shown in Fig. 6.4.2, subject to the following design action:

Axial Compression $N_c^* = 70$ kN

The attached brace member is a Grade 250 100x100x10 EA with a design member capacity in axial compression of $\phi N_c = 100$ kN.

Note: Single angle brace members are not commonly specified for compression braces. However, an angle is used in this example to illustrate the methodology of the design model.

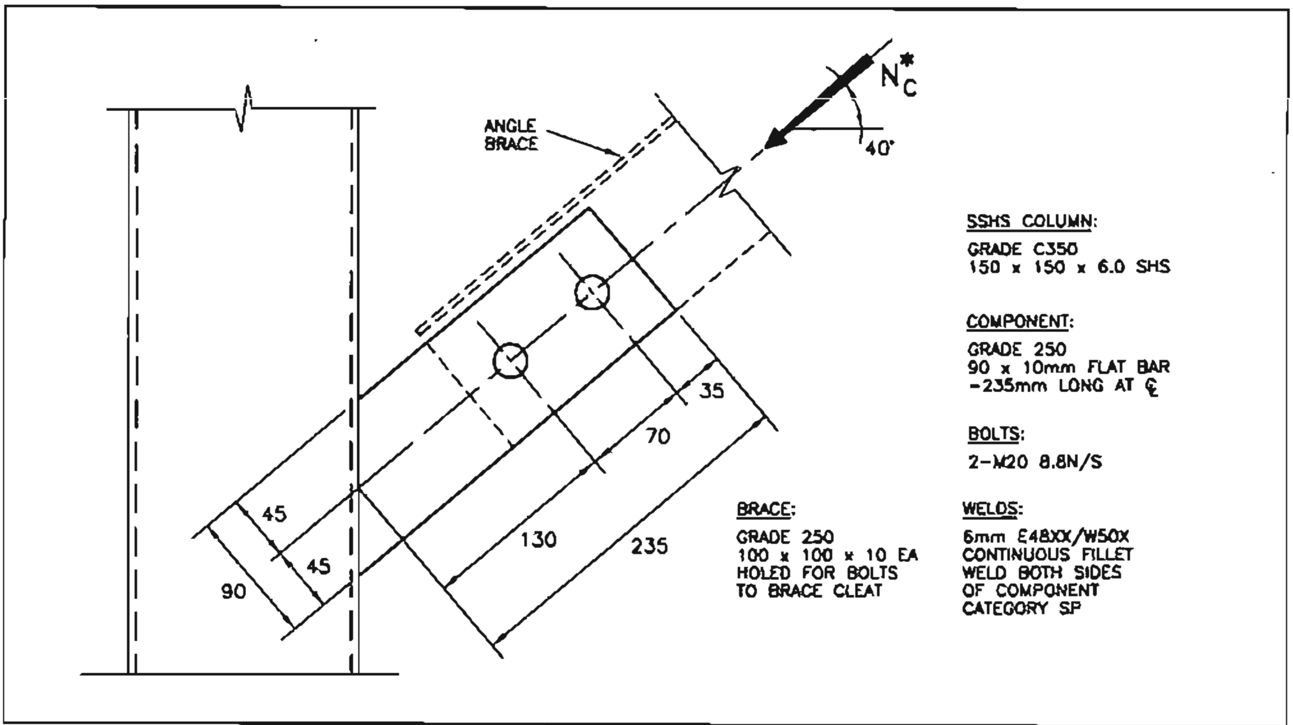

Fig. 6.4.2: Bracing Cleat Geometry

Connection Geometry

d_p = 235 mm
b_p = 90 mm
t_p = 10 mm
d = 150 mm
b = 150 mm
t = 6.0 mm
θ = 40°

s_p = 70 mm
s_g = 0 mm
s_o = 130 mm
a_{e1} = 35 mm
$a_{e2} = 70 - \dfrac{22}{2} = 59$ mm
a_{e3} = 45 mm

n_p = 2
n_g = 1
n_b = 2
d_f = 20 mm
d_h = 22 mm
t_w = 6 mm
$b_v = \dfrac{90}{\cos 40°} = 117$ mm

f_y = 350 MPa
f_u = 430 MPa
f_{yp} = 260 MPa
f_{up} = 410 MPa
f_{uw} = 480 MPa

Minimum Design Action Effects

For Grade 250 100x100x10 EA brace member:

$\phi N_c = 100$ kN			
$0.3\,\phi N_c = 0.3 \times 100$	$= 30$ kN		Note 6 of 6.1
	$< N_c^* = 70$ kN	Satisfactory	

$\therefore\ N_c^* = 70$ kN is adequate.

Design Capacity of the Connection

(i) Column wall shear capacity to Section 6.3.1.1

$$N_a = \frac{2 \times 0.9 \times 0.6 \times 350 \times 117 \times 6.0 \times 10^{-3}}{\sin 40°} = 413 \text{ kN} \qquad 6.3.1.1$$

$> N_c^*$ Satisfactory

(ii) Column face yielding capacity to Section 6.3.1.2

Check SSHS column face slenderness validity range:

$$\frac{b}{t} = \frac{150}{6.0} = 25$$

< 30 Satisfies validity range 6.3.1.2

$$\beta = \frac{10}{150} = 0.0667$$

$$\eta = \frac{117}{150} = 0.780$$

$$\phi N_H = \frac{350 \times 6.0^2}{(1 - 0.0667)} \times \left(2 \times 0.780 + 4 \times \sqrt{1 - 0.0667}\right) \times 10^{-3} = 73.2 \text{ kN}$$

$$N_b = \frac{73.2}{\cos 40°} = 95.6 \text{ kN} \qquad 6.3.1.2$$

$> N_c^*$ Satisfactory

(iii) Component (bolt) bearing capacity to Section 6.3.2.1

$N_c = 2 \times 0.9 \times 3.2 \times 20 \times 10 \times 410 \times 10^{-3} = 472$ kN 6.3.2.1

$> N_c^*$ Satisfactory

(iv) Component (bolt) tearout capacity to Section 6.3.2.2

$N_d = 2 \times 0.9 \times 59 \times 10 \times 410 \times 10^{-3} = 435$ kN 6.3.2.2

$> N_c^*$ Satisfactory

(v) Component buckling capacity to Section 6.3.2.3

$$L_e = 0.7 \times 130 \qquad = 91 \text{ mm}$$

$$r = \sqrt{\frac{10^2}{12}} \qquad = 2.89$$

$$\lambda_n = \left(\frac{91}{2.89}\right) \times \sqrt{1.0} \times \sqrt{\frac{260}{250}} \qquad = 32.1$$

$$\alpha_a = \frac{2100 \times (32.1 - 13.5)}{32.1^2 - 15.3 \times 32.1 + 2050} \qquad = 15.1$$

$$\lambda = 32.1 + 15.1 \times 0.5 \qquad = 39.7 \text{ mm}^2$$

$$\eta = 0.00326 \times (39.7 - 13.5) \qquad = 0.0854 \ \geq 0$$

$$\xi = \frac{\left(\frac{39.7}{90}\right)^2 + 1 + 0.0854}{2 \times \left(\frac{39.7}{90}\right)^2} \qquad = 3.29$$

$$\alpha_c = 3.29\left\{1 - \sqrt{\left[1 - \left(\frac{90}{3.29 \times 39.7}\right)^2\right]}\right\} \qquad = 0.906$$

$$N_s = 1.0 \times (90 \times 10) \times 260 \times 10^{-3} \qquad = 234 \text{ kN}$$

N_e $= 0.9 \times 0.906 \times 234$ $= 191$ kN $> N_c^*$ Satisfactory 6.3.2.3

Note: A short cut to determining α_c would be to calculate λ_n and use Table 6.3.3(3) of AS 4100 with $\alpha_b = 0.5$. By linear interpolation with $\lambda_n = 32.1$:

$$\alpha_c = \frac{(0.891 - 0.917)}{(35 - 30)} \times (32.1 - 30) + 0.917 \qquad = 0.906$$

T6.3.3(3) of AS 4100

(vi) Weld design to Section 6.3.3

$\phi v_w = 0.978$ kN/mm T2.3.3.2-1

N_f $= 2 \times 0.978 \times 117$ $= 229$ kN $> N_c^*$ Satisfactory 6.3.3

(vii) Bolt design to Section 6.3.4

$\phi V_{fn} = 92.7$ kN T2.3.4.4-2

N_i $= 2 \times 92.7$ $= 185$ kN $> N_c^*$ Satisfactory 6.3.4

EXAMPLE 3: BRACING CLEAT IN AXIAL COMPRESSION CONNECTED TO A SSHS BRACE MEMBER

Design Parameters

The compression angle brace member in Example 2 is replaced by a Grade C350 65x65x5.0 SHS with welded tee end (Example 2 of Section 8.4) and a design member capacity in axial compression of ϕN_c = 150 kN.

Connection Geometry

Same as in Example 2 above.

Minimum Design Action Effects

For Grade C350 65x65x5.0 SHS brace member

$\phi N_c = 150$ kN			
$0.3\,\phi N_c = 0.3 \times 150$	$= 45$ kN		Note 6
	$< N_c^* = 70$ kN	Satisfactory	of 6.1

$\therefore\ N_c^* = 70$ kN is adequate.

Design Capacity of the Connection

(i) Column wall shear capacity to Section 6.3.1.1

$N_a = 413$ kN	(see (i) in Example 2)		6.3.1.1
$> N_c^*$		Satisfactory	

(ii) Column face yielding capacity to Section 6.3.1.2

$N_b = 95.6$ kN	(see (ii) in Example 2)		6.3.1.2
$> N_c^*$		Satisfactory	

(iii) Component (bolt) bearing capacity to Section 6.3.2.1

$N_c = 472$ kN	(see (iii) in Example 2)		6.3.2.1
$> N_c^*$		Satisfactory	

(iv) Component (bolt) tearout capacity to Section 6.3.2.2

$N_d = 435$ kN	(see (iv) in Example 2)		6.3.2.2
$> N_c^*$		Satisfactory	

(v) Eccentrically connected cleat component capacity to Section 8.3.2.3

$N_e = 112$ kN	(see (iii) in Example 2 of Section 8.4)		8.3.2.3
$> N_c^*$		Satisfactory	

(vi) Weld design to Section 6.3.3

N_f	$= 229$ kN	(see (vi) in Example 2)		6.3.3
	$> N_c^*$		Satisfactory	

(vii) Bolt design to Section 6.3.4

N_i	$= 185$ kN	(see (vii) in Example 2)		6.3.4
	$> N_c^*$		Satisfactory	