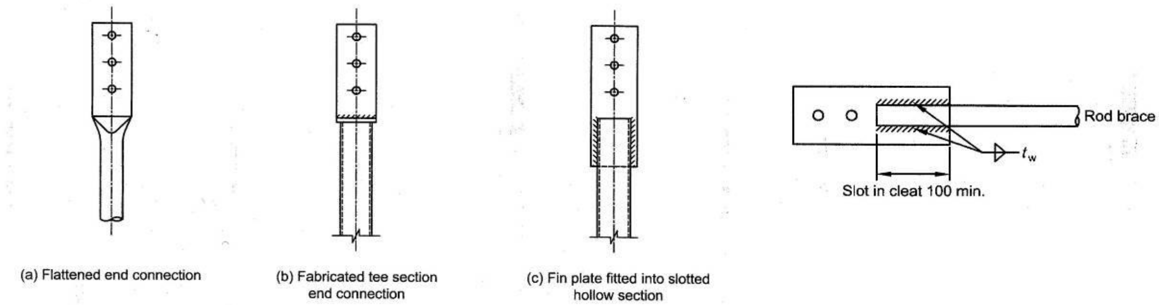
Light Brace Cleat Connections for Braced Steel Frames

Author: *Alistair Fussell*
Affiliation: *Steel Construction New Zealand Inc.*
Date: *25th May 2011*
Ref: *CON1302*

Key Words
Cleat plates, concentric connections, eccentric cleat connections, gusset plates

Introduction

Light bracing cleat connections are defined as unstiffened cleats that connect light bracing such as flat bars, rods or small tubes sections (round or square) to beams or columns (figure 1). These types of bracing systems are typically used in low rise industrial and retail buildings as roof and wall bracing (Hogan and Collins, 2010).



(a) Small tube bracing **(b) Rod bracing**

Figure 1: Examples of Light Braces and Their End Connections (Hogan and Collins, 2010)

Light bracing cleats may be of several forms; one bracing member connected, two bracing members connected and three bracing members connected, figure 2. These cleats may be subject to tensile or compressive forces and there is typically eccentricity between the connected cleats (centroids of cleat minor axes do not coincide), see figure 3.

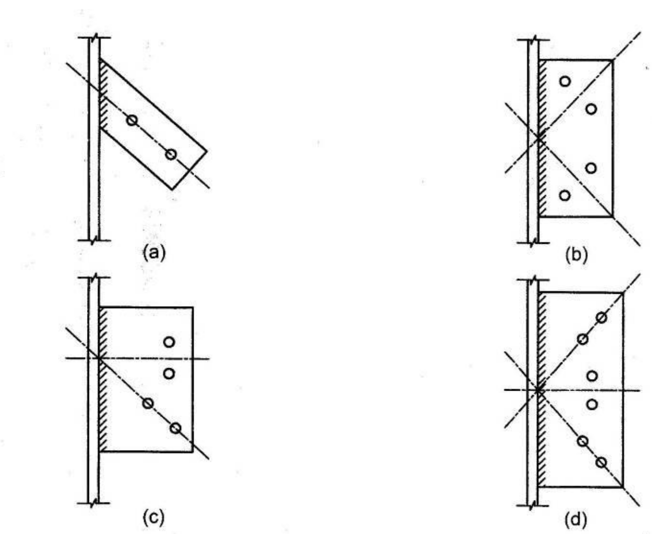


Figure 2: Cleat arrangements for One, Two and Three Member Connections (Hogan and Collins, 2010)

The principle source of design guidance presented in this paper is taken from the ASI Steel Construction Journal article, Design Model for Light Bracing Cleat Connection (Hogan and Collins, 2010). The major area of departure from the guidance presented in (Hogan and Collins, 2010) is the procedure for designing eccentric connections for compression. In this instance reference has been made to HERA report R4-142 (Clifton and El Sarraf, 2007).

This paper does not include the design of the end connection of hollow steel sections such as fin plates slotted into tubes or welded to tube cap plates (figure 1). Such guidance is found in (Syam and Chapman, 1996).

Design Issues Bracing Cleat Plates

Compliance with NZS 3404 Steel Structures Standard

The design provisions that relate to connections are found in section 9 and 12 of NZS 3404 (SNZ, 1997). Section 9 covers general design requirements while section 12 covers the additional requirements for seismic applications. Note the Steel Structures Standard is currently under revision. When published the relevant parts will be NZS 3404.3 Member and connection design and NZS 3404.7 Seismic design.

Minimum design actions

Connections must have a minimum capacity to ensure they are sufficiently robust in the eventuality that a bracing system which theoretically has very small design actions is subject to loads greater than anticipated.

The requirement for minimum design actions for non-seismic applications is:

> *Connections at the ends of tension or compression members – a force of 0.3 times the member design capacity, except that for threaded rod acting as a bracing member, the minimum tension force shall be equal to the member design capacity.*

For connections which are part of a seismic loading resisting must be designed for a minimum design action of:

> *50% of the design section capacity of the member is compression or tension as appropriate (0.5 ϕ N_s or 0.5 ϕ N_t).*

Connection Eccentricity

It is not uncommon in light cleat connections to have eccentricity with respect to the minor axes of the cleats. This could include a flat bar brace connected to one side of a cleat or a hollow section brace fin plate connected to one side of a cleat , see figure 3.

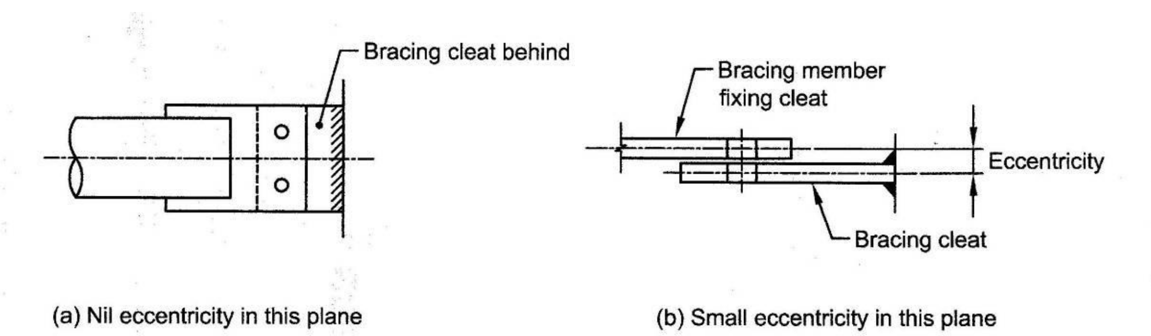


Figure 3: Eccentricity of Cleat Connection (Hogan and Collins, 2010)

This eccentricity with respect to cleat minor axes may be ignored for tension only connections but must be considered for eccentric connections with compressive loading (Hogan and Collins, 2010). It is important to appropriately account for the sway mode of behaviour that governs the design capacity of eccentrically connected cleat connections subject to compression loading. Up until recently the design model used in Australia and New Zealand to design eccentrically connected cleat plates in compression was based on the AISC paper Eccentrically Connected Cleat Plates in Compression, by Kitipornchai, Al-Bermani and Murray (1993). This procedure reportedly addressed the issue of connection eccentricity.

Concerns were raised several years ago in Australia over the use of the model which failed to recognise the governing mode of behaviour as a sway failure mode. Following these concerns Clifton and El Sarraf (2007) proposed a design model utilising the combined bending and compression provisions of NZS 3404 with appropriate account taken of second order effects, fabrication tolerances and sway mode behaviour (figure 4).

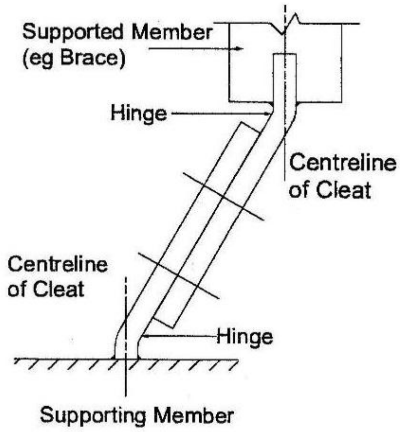


Figure 4: Sway Mode behaviour of Eccentric Cleat Connection in Compression (Clifton and El Sarraf, 2007)

The full step by step procedure is presented in Clifton and El Sarraf (2007). The salient points of the methodology are as follows:

1. It is not applicable to connections involving category 1,2 and 3 primary members in seismic load resisting systems i.e. not applicable for concentrically braced frames with braces effective in compression and tension. However, it could be used for compression member connections which are secondary members in tension only concentrically braced systems.
2. There is a maximum gravity load limit for seismic applications $0.7\phi N_c \geq N^*_{G+Q_u}$
3. No load restriction applies to connections subject to non-seismic loading.
4. The cleat is treated as a column designed for combined actions in accordance with NZS 3404. As a result of this sway behaviour the connections and brace members are subject to bending in addition to axial loading.
5. The procedure is applicable to unstiffened cleats and for the case where one of the cleats is stiffened
6. A sway mode is the predominant failure mode.
7. At least one of the cleats in a connection must be fixed against rotation to provide resistance to lateral movement of the of the joint due to sway mode behaviour (figure 5).
8. An additional 3mm eccentricity is allowed in the connection to account for fit up tolerances.
9. The first order moments due to eccentricity must be magnified by a sway factor δ_s. Guidance is provided on how to apply the second order provisions of NZS 3404 to an eccentrically connected cleat connection. An upper limit δ_s of 1.33 is set. If this criterion is not met one of the cleats must be stiffened.
10. The amplified joint moment must be less than the sum of the minor moment capacity of the connected cleat reduced for axial loading. Note the elastic rather than the plastic section modulus is used to to calculate M_{sy} as the connection stiffness is very sensitive to plate yielding.

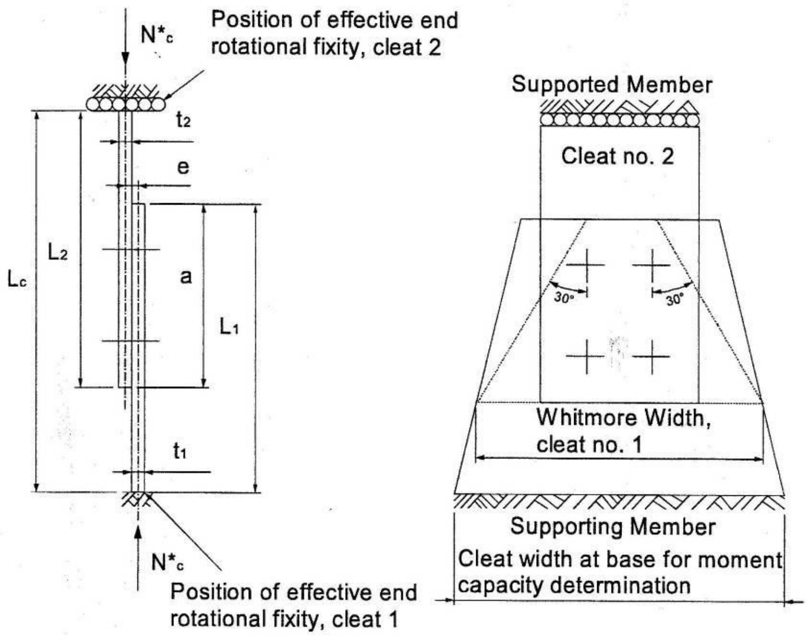


Figure 5: Eccentric Cleat in Compression Design Model (Clifton and El Sarraf, 2007)

Gusset Plates with Multiple Connections

Introduction

Gusset plates with multiple connections represent a more complex situation than an isolated cleat. A feature of gusset plate connections is that the size of the plate is typically large compared to the bolted or welded joints within them. The approach taken to this situation is to treat each brace as connected to an individual bracing cleat whose width is defined as the Whitmore section (Hogan, 2010), see figure 6.

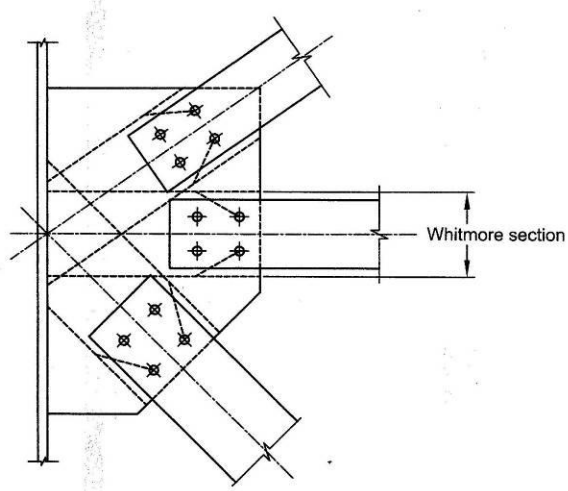


Figure 6: Application of Whitmore Width to Multiple Member Gusset Plates (Hogan and Collins, 2010)

There is a very extensive treatment of gusset plate design and behaviour in section 10.7.2 of the HERA Structural Steelwork Limit State Design Guide Volume 1 (Clifton, 1994). Note the block shear design equations presented in (Clifton 1994), which were taken from the Manual of Steel Construction (AISC, 1995), have now been revised. This is discussed below.

Whitmore Section

The Whitmore section is defines as the length of line taken through the last line of fasteners (or end of weld line) and extending to the intersection of the lines drawn from the first line of fasteners (or start of weld line) at an angle of 30 degrees from the line of fasteners (or welds) (Clifton 1994). The construction of the Whitmore section is shown in figure 7.

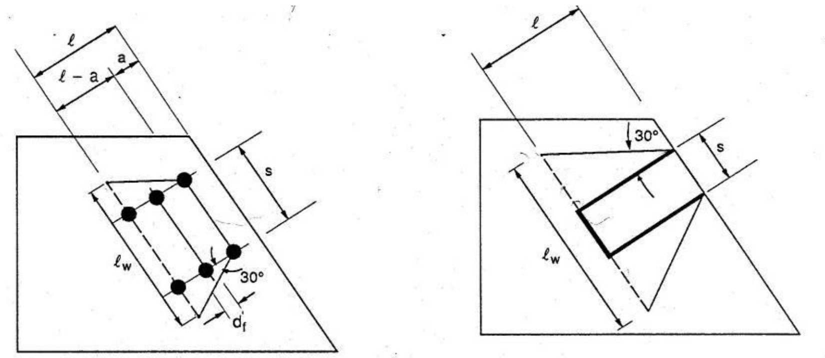


Figure 7: Construction of Whitmore Section (Clifton, 1994)

For multiple member connections there is the possibility that the Whitmore widths as described may over lap. In this instance the width should be limited to avoid such an overlap (Hogan and Collins, 2010).

The Whitmore width concept is used to check both the yield capacity of the plate under tension and also the buckling capacity under compressive loading. Note the Whitmore width as described must be modified for eccentrically connected cleat connections using the methodology of (Clifton and El Sarraf, 2007). The methodology for checking the buckling capacity of a concentrically connected gusset plate connection is discussed below under plate limit states.

Combined Actions on Plate

Depending on the connection geometry and design loads, the gusset plate and connecting welds can be subject to resultant shear, axial and flexural design actions. These can determined by consider the resultants of the design loads with respect to the plate and weld centroids, see figure 8.

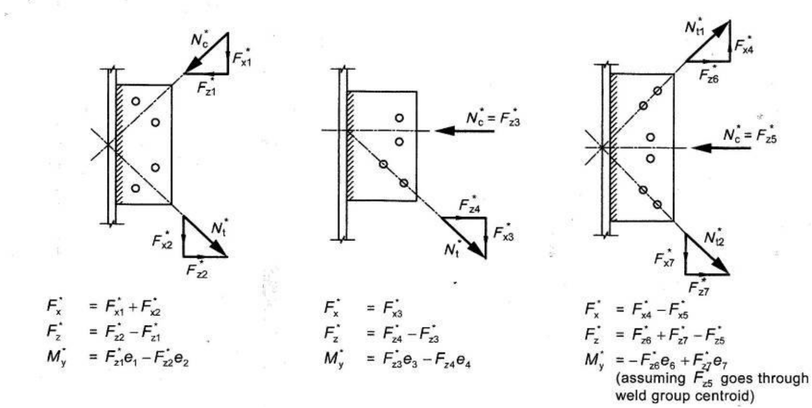


Figure 8: Resultant Design Actions on Multiple Brace Gusset Plates (Hogan and Collins, 2010)

The brace forces can be resolved into normal and transverse components F_x and F_z. In this instance F_x actions give rise to shear forces in the plate while F_z components give rise to axial forces. A resultant moment M_y is present if there is eccentricity between F_x and F_z relative to the weld or plate centroids.

Plate combined actions can be checked using equation 10.7 from (Clifton, 1994).

$$f_x^{*2} + f_y^{*2} - f_x^* f_y^* + 3\tau_{xy}^{*2} \le \left(\phi f_y\right)^2$$

where:

f_x^* design normal stress in x direction (see figure 9)

f_y^* design stress in y direction (typically zero)

τ_{xy}^* design shear stress in xy plane

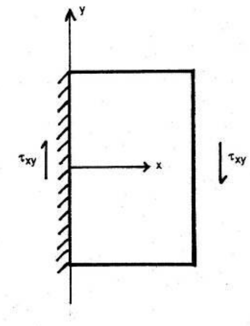


Figure 9: Generalised Yielding (Clifton, 1994)

When moment is present, f_x^*, is the summation of normal stresses due to bending and axial loading. Calculation of f_x^* due to bending can be undertaken using plastic or elastic properties. Assuming elastic properties will cover all likely situations (Clifton, 1994).

Connection Limit States

Introduction

The design capacity of a given connection will be the limit state or failure mode with the lowest capacity. This will include not only the various limit states of the plate such as yielding, buckling, rupture (fracture and block shear) and plate tearing adjacent to bolts (figures 10 and 11) but also the other components such as welds and bolts. A final consideration is the design capacity of the supporting member locally at cleat location.

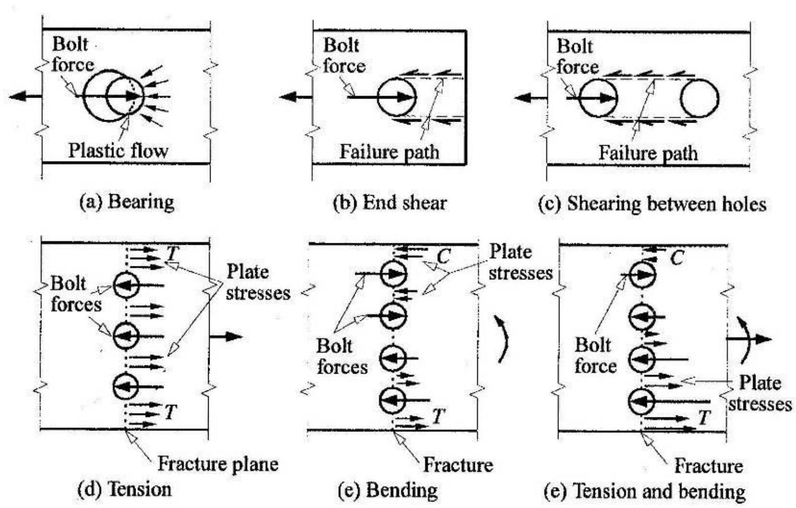


Figure 10: Limit States for Bolted Plate Connections (Trahair and Bradford, 1998)

Cleat Plate Yielding

The cleat yield capacity is based on the gross area of the cleat for narrow cleats similar to the width of the bolted or welded connection, or the Whitmore section area if the plate is wide compared to the bolted or welded connection. The Whitmore width is discussed above.

$$\phi N_{ty} = \phi A_g f_y$$

where:

A_g = plate gross area for narrow cleat, or Whitmore section
f_y = yield stress of plate

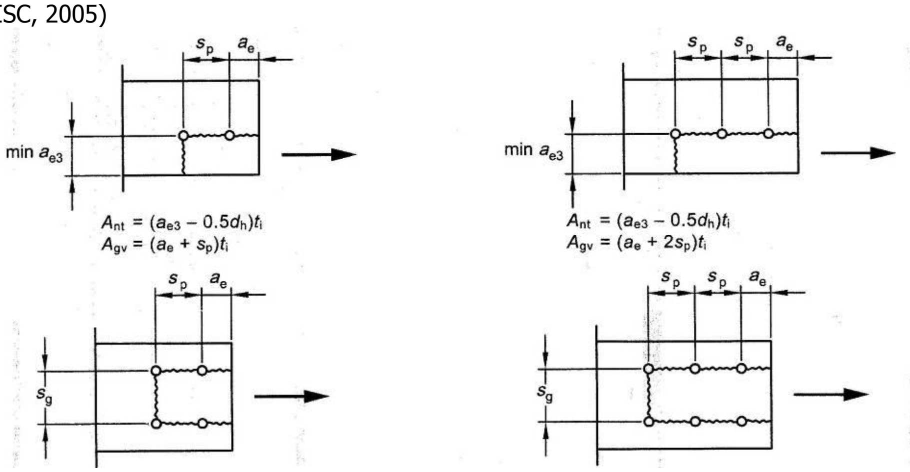
Cleat Plate Fracture
The cleat design capacity limited by fracture through the net section is calculated as follows:

$\phi N_{tf} = \phi 0.85 A_n f_u$

f_u = tensile strength
A_n = net area allowing for holes

Cleat Plate Block Shear
There are currently no block shear provisions in NZS 3404. Block shear equations are presented in the AISC Manual of Steel Construction (2005).

Block shear failure involves both shear and tensile failure. Examples of block shear failure for isolated cleats are shown in figure 11. Block shear failure should also be checked around the periphery of welded connections (AISC, 2005)



$A_{nt} = (S_g - d_h))t_i$
$A_{gv} = 2(a_e + s_p)t_i$

$A_{nt} = (S_g - d_h))t_i$
$A_{gv} = 2(a_e + 2s_p)t_i$

Figure 11: Examples of Block Shear Failure for Isolated Cleats (Hogan and Collins, 2010)

Block shear is a rupture or tearing phenomenon, not a yielding limit state. However, gross yielding on the shear plane can occur when tearing on the tensile plane commences if $0.6 f_u A_{nv}$ exceeds $0.6 f_y A_{gv}$.(AISC 2010).

$$\phi N_{bs} = \begin{bmatrix} \phi(0.6 f_y A_{gv} + f_u A_{nt}) \\ \phi(0.6 f_u A_{nv} + f_u A_{nt}) \end{bmatrix}_{min}$$

Where:

f_u	=tensile strength of cleat
f_{yi}	=yield stress of cleat
A_{nt}	=net area subject to tension
A_{nv}	=net shear area
A_{gv}	=gross area subject to shear

Gusset Plate Buckling
a. Concentric Cleat Connection (with respect to cleat minor axes)

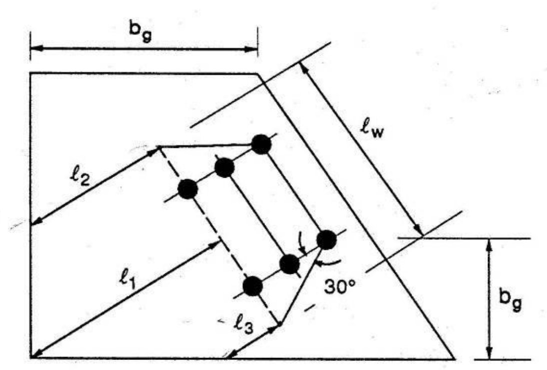


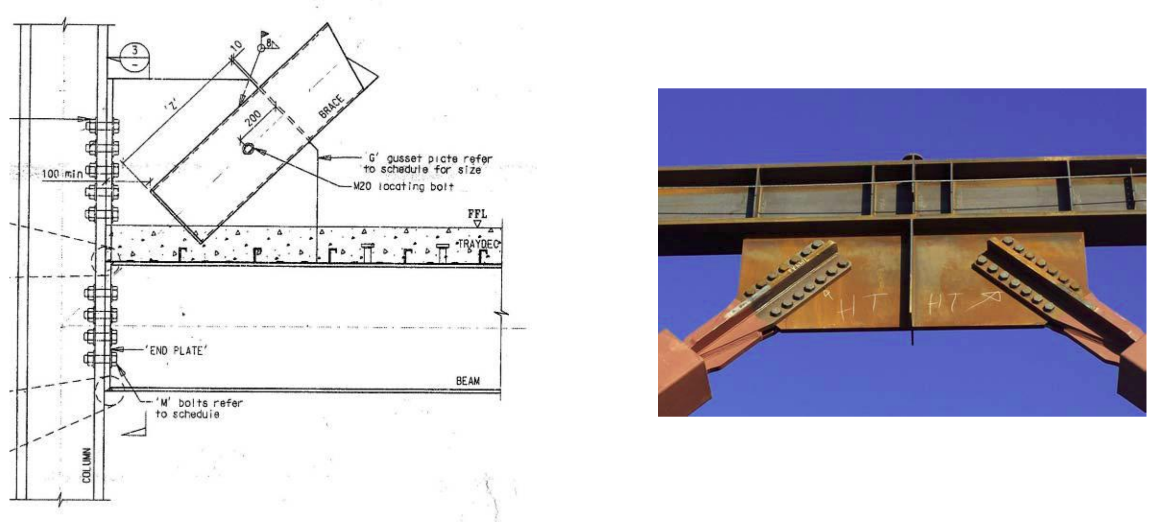
Figure 12 Parameters for Computing Gusset Plate Buckling Capacity (Clifton, 1994)

The effective length used to compute the design compression capacity of the Whitmore section is found by treating it as a column and using the nominal member capacity provisions of NZS 3404 in the following manner:

1. The design length L of the column is taken as the average of l_1,l_2 and l_3 (figure 12).
2. The appropriate effective length factor k_e is dependent on the gusset plate configuration.
   a. k_e=0.7 corner gusset plates (figure 13)
   b. k_e=1.4 midspan gusset plates (figure 13)
3. The radius of gyration of the plate $r = \dfrac{t_p}{\sqrt{12}}$
4. The form factor, k_f is taken as 1.0 and the member constant α_b=0.5 for use in clause 6.3.3
5. $N_s=A_{ww}f_yk_f$

where:

A_{ww} Area of Whitmore section
t_p Gusset plate thickness



(a) corner gusset plate **(b) midspan gusset plate**

Figure 13: Examples of Corner and Mid-span Gusset Plate Connections

The effective length factor for corner gusset plate connections is specified in NZS 3404. The effective length factor for midspan gusset plates is taken from Roeder and Lehman (2008). Experiment work by Roeder and other researchers has shown that calculating the gusset plate buckling capacity using an effective length determining using the method for corner gusset plates is non conservative.

b. Eccentric Cleat Connections
The design model used for eccentrically connected cleat connections is discussed above.

Plate Combined Stress
Check plate stresses using general yielding equation from combine actions on plate section above.

Gusset Plate In-plane Stability of Unsupported Compression Edges.
The gusset plate unsupported edge, b_g (figure 12), should be checked against local buckling under compression loading using equation 10.16 from (Clifton, 1994).

$$b_g \leq C_1 t_p / \sqrt{(f_y / 250)}$$

where:

C_1 = 45 for gusset plates bolted to the supported member(s)
C_1 = 40 for gusset plates welded to the supported member(s)

Bolt Limit States
The general requirements for bolts are given in section 9 of NZS 3404. The appropriate bolt limit states for cleat connections are:

1. The shear capacity of a bolt is dependent amongst other things on the number of shear planes (single or double) or whether the threads are included or excluded from the shear plane. See table 1 for shear values for grade 8.8 bolts. The shear capacity of a long bolted connection is reduced due to non uniform load distribution. A reduction factor applies for bolted joints where the centreline dimension between the first and last bolt is greater than 300mm (table 9.3.2.1 NZS 3404)
2. Ply in bearing
   a. Local bearing failure $\phi V_b = \phi 3.2 d_f t_p f_{up}$ (see figure 10)
   b. End plate tearout $\phi V_b = \phi a_e t_p f_{up}$ (see figure 10)

where:

d_f is the bolt diameter
t_p is the plate thickness
f_{up} is the plate tensile strength
a_e is the bolt edge distance in the direction of loading (see figure 11)

Table 1: Grade 8.8 Bolt Capacities (AISC, 1999)

Bolt Size	Axial Tension	Single Shear		Plate Tearout in kN												Bearing in kN		
		Threads included in Shear Plane	Threads excluded from Shear Plane	ϕV_b for t_p & a_e of:												ϕV_b for t_p		
				$t_p = 6$			$t_p = 8$			$t_p = 10$			$t_p = 12$			6	8	10
	ϕN_{tf}	ϕV_{fn}	ϕV_{fx}															
	kN	kN	kN	35	40	45	35	40	45	35	40	45	35	40	45			
M16	104	59.3	82.7													122	162	203
M20	163	92.6	129	83	95	107	111	127	143	139	158	178	166	190	214	152	203	253
M24	234	133	186													182	243	304
M30	373	214	291													228	304	380
M36	541	313	419	$a_e > a_{emin} = 1.5 d_f$												274	355	456
	$\phi = 0.8$	$\phi = 0.8$		$\phi = 0.9$												$\phi = 0.9$		
		8.8N/S	8.8X/S	f_{up} = 440 MPa												f_{up} = 440 MPa		

The f_u values appropriate with various grades of plate are as follows:

Table 2: Grade Dependent f_u Values

Grade	f_{up} MPa
250	410
300	430
350	450
400	480

Note: f_u is plate thickness dependent. Therefore, for plate thicker than 16mm reference should be made to the appropriate plate standards (AS/NZS 3678).

Additional values of end plate tearing are given in table T9.5 from (AISC, 1999).

Weld Limit States
Cleat connections are subject to a combination of resultant design actions depending on the arrangement of braces. The weld may either be a full strength butt weld or a weld group consisting of two parallel lines of fillet welds, figure 14. The equations that govern the design of this fillet weld group are as follows.

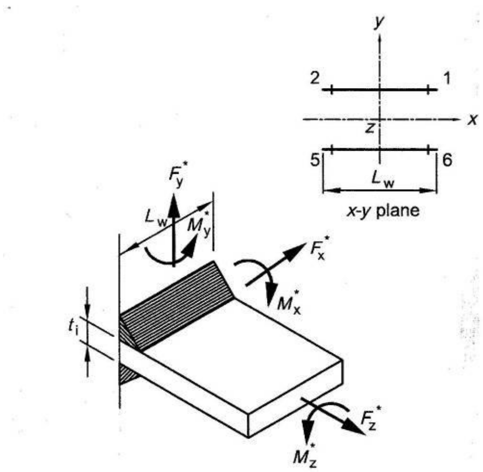


Figure 14: Design Actions on Parallel Weld Group (Hogan and Collins, 2010)

The weld group properties are

$L_{wx}=L_{wy}=L_{wz}=2L_w$

$I_{wy} = (L_w^3)/6$

At points 1,2,5 and 6 y=t/2

The design forces per unit length are:

$V^*_x=F^*_x/(2L_w)$

$V^*y=F^*_y/(2Lw)$

$$V^*_z=\frac{F_z^*}{2L_w}+\frac{M_x^*}{L_w t}+\frac{M_y*\left(L_w/2\right)}{I_{wy}}$$

The governing equation for fillet welds is: $\sqrt{\left((v^*_x)^2+(v^*_y)^2+(v^*_z)^2\right)} \le \phi v_w$

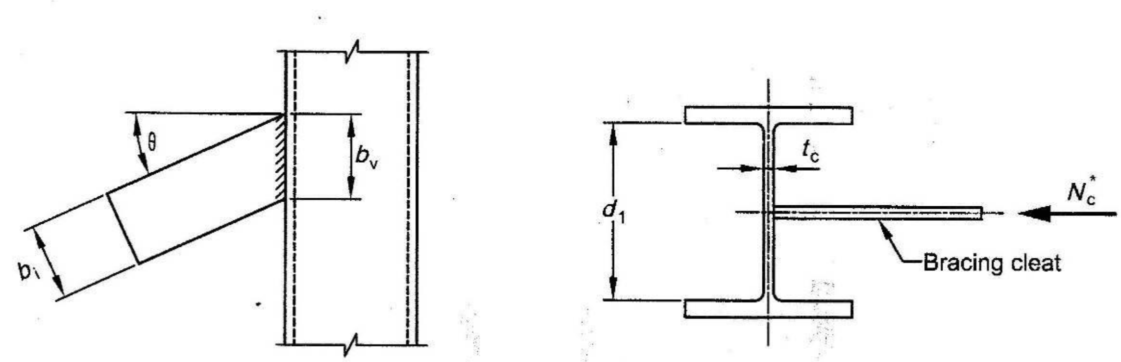
The following tabulated values for the nominal capacity of fillet weld per length are taken from the AISC Design Capacity Tables for Structural Steel (1999).

Table 3: Fillet Weld Capacities SP Welds (AISC, 1999)

Weld Size (mm)		Design Capacity per unit length of weld, ϕv_w (kN/mm)	
t_w	t_t	E41XX/W40X	E48XX/W50X
3	2.12	0.417	0.489
4	2.83	0.557	0.652
5	3.54	0.696	0.815
6	4.24	0.835	0.978
8	5.66	1.11	1.30
10	7.07	1.39	1.63
12	8.49	1.67	1.96
		f_{uw} = 410 MPa	f_{uw} = 480 MPa

Support Member Locally at Cleat Location Limit State

Depending on the location of the bracing cleat to the supporting beam or column, the supporting elements may be subject to local design actions such as bending of the webs of I sections or the faces of circular or rectangular hollow sections (figure 15).



(a) Cleat to face of hollow section **b) cleat connected mid web height**

Figure 15: Cleat Positions Inducing Local Design Actions in Beam/Column Elements (Hogan and Collins, 2010)

Solutions for the support member design actions induced by bracing cleats connected to the faces of hollow sections or the webs of I sections are presented in (Hogan and Collins, 2010).

References

AISC, Specification for Structural Steel Buildings, ANSI/AISC 360-10, American Institute of Steel Construction , Chicago, Illinois, 2010

AISC, Design Capacity Tables for Structural Steel, 3rd Edition, Australian Institute of Steel Construction, Sydney, 1999

Clifton, C. and El Sarraf, R., Eccentric Cleats in Compression and Columns in Moment Resisting Connections, NZ Heavy Engineering Research Association, Manukau City, 2007

Clifton, G.C., New Zealand Structural Steel Limit State Design Guide Volume 1, NZ Heavy Engineering Research Association, Manukau City, 1994

Hogan, T.J. and Collins, R.T., Design Model for Light Bracing Cleat Connections, Australian Institute of Steel Construction, volume 43 Number 2, Sydney, March 2010

Hogan, T.J, and Munter, S.A., Connection Handbook 1 – Background and Theory, Australian Steel Institute, Sydney, 2007

Kitipornchai, S., Al-Bermani, F.D.A. and Murray, N.W, Eccentrically Connected Cleat Plates in Compression, Journal of Structural Engineering, ASCE, volume 11, No 3, 1993

Roeder, C., W., and Lehman, D., E., Performance Behaviour of Gusset Plate Connections, Conference proceedings, AISC North Americas Steel Conference, Phoenix, Arizona, April 2008

Syam, A.A and Chapman, B.G., Design of Structural Hollow Section Connections, Australian Institute of Steel Construction, Sydney, 1996

SNZ, Steel Structures Standard: NZS 3404 Parts 1 and 2, Standards New Zealand, Wellington, 1997

Trahair, N.S. and Bradford, M.A., The Behaviour and Design of Steel Structures to AS 4100, 3 rd edition, E & FN Spon, London, 1998