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).

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

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)](image)

Disclaimer: SCNZ and the author(s) of this document make no warrantee, guarantee or representation in connection with this document and shall not be held liable or responsible for any loss or damage resulting from the use of this document.
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 $\phi N_s$ or 0.5 $\phi N_t$).*

**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.

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).
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 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 \(\delta_s\). Guidance is provided on how to apply the second order provisions of NZS 3404 to an eccentrically connected cleat connection. An upper limit \(\delta_s\) 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 calculate \(M_{sy}\) as the connection stiffness is very sensitive to plate yielding.
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.

Whitmore Section
The Whitmore section is defined 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)](image)

For multiple member connections there is the possibility that the Whitmore widths as described may overlap. 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 be determined by considering 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)](image)

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).

$$f_x^2 + f_y^2 - f_x f_y + 3f_y^2 \leq (\phi f_y)^2$$

where:
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.

\[
\phi N_{fy} = \phi A_g f_y
\]

**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.
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:

\[
\phi N_d = \phi 0.85 A_n f_u
\]

\( f_u \) = 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)

\[
A_{nt} = (a_{n3} - 0.5d_h) t_i
\]

\[
A_{gv} = 2(a_{v} + s_v) t_i
\]

**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).

\[
\phi N_{bs} = \left[ \phi(0.6 f_y A_{gv} + f_u A_{nt}) \right]_{\text{min}}
\]

Where:

\( f_u \) = tensile strength of cleat

\( f_y \) = 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)*

![Diagram of Gusset Plate Buckling](image)

**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.
   - \(k_e=0.7\) corner gusset plates (figure 13)
   - \(k_e=1.4\) midspan gusset plates (figure 13)
3. The radius of gyration of the plate \(r = \frac{t_p}{\sqrt{12}}\)
4. The form factor, \(k_f\) is taken as 1.0 and the member constant \(\alpha_b=0.5\) for use in clause 6.3.3
5. \(N_s = A_{ww}f_y k_f\)

where:

- \(A_{ww}\) Area of Whitmore section
- \(t_p\) Gusset plate thickness

![Images of Gusset Plate Connections](image)

(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 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).

\[
b_g \leq C\frac{t_p}{\sqrt{f_y/250}}
\]

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 \( \phi V_b = \phi 3.2 d_t f_{up} \) (see figure 10)
   b. End plate tearout \( \phi V_b = \phi a_e t_p f_{up} \) (see figure 10)

where:

\( d_t \) 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)

<table>
<thead>
<tr>
<th>Bolt Size</th>
<th>Axial Tension</th>
<th>Single Shear</th>
<th>Plate Tearout in kN</th>
<th>Bearing in kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \phi N_f )</td>
<td>( \phi V_n )</td>
<td>( \phi V_b ) for ( t_p ) &amp; ( a_e ) of:</td>
<td>( \phi V_b ) for ( t_p )</td>
</tr>
<tr>
<td></td>
<td>kN</td>
<td>kN</td>
<td>35 40 45 35 40 45 35 40 45 35 40 45 35 40 45</td>
<td>6 8 10</td>
</tr>
<tr>
<td>M16</td>
<td>104</td>
<td>59.3</td>
<td>82.7</td>
<td>122 162 203</td>
</tr>
<tr>
<td>M20</td>
<td>163</td>
<td>92.6</td>
<td>129</td>
<td>83 107 111 127 143 139 158 178 186 190 214</td>
</tr>
<tr>
<td>M24</td>
<td>234</td>
<td>133</td>
<td>186</td>
<td>152 203 253</td>
</tr>
<tr>
<td>M30</td>
<td>373</td>
<td>214</td>
<td>291</td>
<td>182 243 304</td>
</tr>
<tr>
<td>M36</td>
<td>541</td>
<td>313</td>
<td>419</td>
<td>226 304 380</td>
</tr>
<tr>
<td></td>
<td>( \phi = 0.8 )</td>
<td>8.8N/S</td>
<td>8.8X/S</td>
<td>( f_{up} = 440 \text{ MPa} )</td>
</tr>
</tbody>
</table>

Steel Advisor CON1302
© Steel Construction New Zealand Inc. 2011
The $f_u$ values appropriate with various grades of plate are as follows:

**Table 2: Grade Dependent $f_u$ Values**

<table>
<thead>
<tr>
<th>Grade</th>
<th>$f_{ud}$ MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>410</td>
</tr>
<tr>
<td>300</td>
<td>430</td>
</tr>
<tr>
<td>350</td>
<td>450</td>
</tr>
<tr>
<td>400</td>
<td>480</td>
</tr>
</tbody>
</table>

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.

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:

$V^*_{x} = F^*_{x} / (2L_w)$

$V^*_{y} = F^*_{y} / (2L_w)$

$V^*_{z} = F^*_{z} / (2L_w) + M^*_{x} / L_w t + M^*_{y} (L_w^2 / 2) / I_{wy}$

The governing equation for fillet welds is:

$\sqrt{(V^*_{x})^2 + (V^*_{y})^2 + (V^*_{z})^2} \leq \phi V_w$
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)**

<table>
<thead>
<tr>
<th>Weld Size</th>
<th>Design Capacity per unit length of weld, ( \phi \nu_w ) per unit length (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( t_w ) (mm)</td>
<td>( t_l )</td>
</tr>
<tr>
<td>3</td>
<td>2.12</td>
</tr>
<tr>
<td>4</td>
<td>2.83</td>
</tr>
<tr>
<td>5</td>
<td>3.54</td>
</tr>
<tr>
<td>6</td>
<td>4.24</td>
</tr>
<tr>
<td>8</td>
<td>5.66</td>
</tr>
<tr>
<td>10</td>
<td>7.07</td>
</tr>
<tr>
<td>12</td>
<td>8.49</td>
</tr>
</tbody>
</table>

\( f_{uw} = 410 \text{ MPa} \quad f_{uw} = 480 \text{ MPa} \)

**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).

![Diagram](image)

(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, 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


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