

Appendix A

Load limit of angular members

From the calculation done for the wind loading on conductors, it was stated that the overturning moment for the double circuit structure is 5.2 times higher compared with a single circuit structure. Thus, to understand what the increase in member loading is from a single circuit tower to a double circuit tower, the following calculation was done. It should also be noted that this calculation only considers the load in the leg members and not any bracing members.

The approach that was used is to firstly calculate the width at the base of the structure for a single circuit and double circuit tower. This was done according to the formulae given by Ryle (1945).

$$W = 0.35\sqrt{M} \quad (\text{A.1})$$

where:

W = width at the tower base

M = overturning moment at the tower base

W_S = relative base width of a single circuit tower

W_D = relative base width of a double circuit tower

Thus;

$$W_S = 0.35\sqrt{1} = 0.35 \quad (\text{A.2})$$

$$W_D = 0.35\sqrt{5.2} = 0.8 \quad (\text{A.3})$$

Now that the relative tower base width is known, the lever arm equation can be used to determine the force in the tower leg member.

$$M = FW \quad (\text{A.4})$$

where:

F = relative force in the tower leg member

F_S = relative force in the tower leg member for a single circuit tower

F_D = relative force in the tower leg member for a double circuit tower

Thus;

$$F_S = \frac{1}{0.35} = 2.86 \quad (\text{A.5})$$

$$F_D = \frac{5.2}{0.8} = 6.5 \quad (\text{A.6})$$

The result show that there is a 127% increase in the leg member load of a double circuit tower compared to a double circuit tower.

To put this into practical terms, consider a typical leg member of a suspension type tower of 150 x 150 x 18. From the *Southern African Steel Construction Handbook* (2008), $C_R = 1040$ kN for $KL = 2000$ mm. With a load increase of 127%, the compressive resistance of the member must be 2360.8 kN. The next size up that is capable of resisting this load is a 200 x 200 x 24 angle section with $C_R = 2390$ at $KL = 1750$ mm.

This leads to the following conclusion:

- The calculation was done for a suspension type structure.
- The increase in leg load member from single circuit tower compared with a double circuit tower is 127%.
- Secondary bracing will be required to reduce the effective length (KL) from 2000mm to 1750mm. This means additional weight.
- There is a member weight (Kg/m) increase of 77%.
- There are no bigger members available beyond 200 x 200 x 24. This means that an alternative is required.
- Circular hollow section sizes go up to 508mm.

Appendix B

Structural capabilities of angle iron members

B.0.1 Slenderness ratio

Next an evaluation of the axial loading capacities of angular tower members will be conducted. Firstly, one needs to view the geometrical properties of an angular cross section.

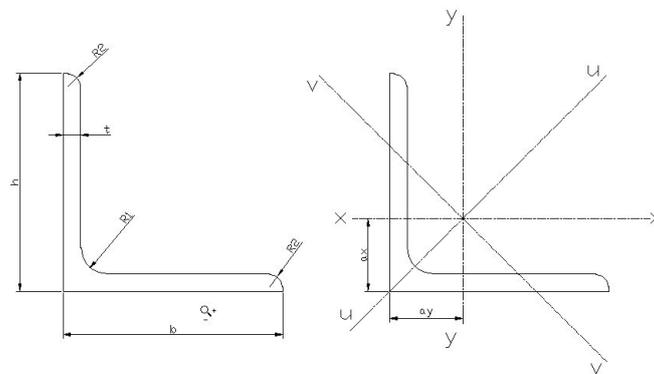


Figure B.1: Angular cross section properties according to *Southern African Steel Construction Handbook* (2008).

From the cross sectional details (figure B.1) it may be seen that there are four axes in which flexural buckling may occur ($x - x$; $y - y$; $v - v$; $u - u$). Assuming that only equal leg members will be considered ($a_x = a_y$). The first critical axis of bending (*Southern African Steel Construction Handbook* 2008) is the $v - v$ axis. The second axis of importance is the $x - x$ ($y - y$) axis.

The question at this point is, why are these axes so important? The answer is that the focus is on the radius of gyration (r) associated with these axes. We know that the radius of gyration has an influence on the slenderness ratio (λ). Thus we need to design around the weakest axis.

The slenderness is calculated with:

$$\lambda = KL/r \quad (\text{B.1})$$

where,

K = effective length coefficient

L = unbraced length

r = radius of gyration

The limiting slenderness ratios (λ) for different tower components e.g. tower leg members, bracing members and redundant members, are specified by *ASCE 10-97:Design of latticed steel transmission structures* (1997) as:

- leg members = 120
- all other members carrying calculated stress = 200
- redundant members = 250

The following slenderness curves have also been defined by *ASCE 10-97:Design of latticed steel transmission structures* (1997):

For members with concentric load at both ends of the unsupported panel:

$$\frac{KL}{r} = \frac{L}{r} \quad 0 \leq \frac{L}{r} \leq 120 \quad (\text{B.2})$$

For members with a concentric load at one end and normal framing eccentricity at the other end of the unsupported panel:

$$\frac{KL}{r} = 30 + 0.75 \frac{L}{r} \quad 0 \leq \frac{L}{r} \leq 120 \quad (\text{B.3})$$

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For members with normal framing eccentricities at both ends of the unsupported panel:

$$\frac{KL}{r} = 60 + 0.5\frac{L}{r} \quad 0 \leq \frac{L}{r} \leq 120 \quad (\text{B.4})$$

For members unrestrained against rotation at both ends of the unsupported panel:

$$\frac{KL}{r} = \frac{L}{r} \quad 120 \leq \frac{L}{r} \leq 200 \quad (\text{B.5})$$

For members partially restrained against rotation at one end of the unsupported panel:

$$\frac{KL}{r} = 28.6 + 0.762\frac{L}{r} \quad 120 \leq \frac{L}{r} \leq 225 \quad (\text{B.6})$$

For members partially restrained against rotation at both ends of the unsupported panel:

$$\frac{KL}{r} = 46.2 + 0.615\frac{L}{r} \quad 120 \leq \frac{L}{r} \leq 250 \quad (\text{B.7})$$

Once $\frac{KL}{r}$ is known, the design compressive stress on the cross-sectional area may be calculated according to *ASCE 10-97: Design of latticed steel transmission structures* (1997) as follows:

$$F_a = [1 - \frac{1}{2}(\frac{KL/r}{C_c})^2]F_y \quad \frac{KL}{r} \leq C_c \quad (\text{B.8})$$

$$F_a = \frac{\pi^2 E}{(\frac{KL}{r})^2} \quad \frac{KL}{r} > C_c \quad (\text{B.9})$$

$$C_c = \pi \sqrt{\frac{2E}{F_y}} \quad (\text{B.10})$$

where,

F_y = minimum guaranteed yield stress

E = modulus of elasticity

In conclusion, CHS members do not have either a weak or a strong axis. The material through the cross section is symmetrical which leads to higher structural efficiency.

B.0.2 Width-to-thickness ratio

Another design consideration is the maximum w/t ratio (width-to-thickness). See figure B.2.

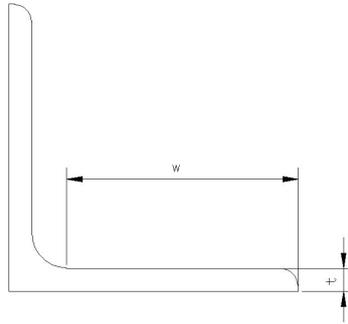


Figure B.2: Determination of w/t .

$$\frac{w}{t} \leq 25 \quad (\text{B.11})$$

When w/t exceeds $(w/t)_{lim}$ as described by *ASCE 10-97:Design of latticed steel transmission structures* (1997) through:

$$\frac{w}{t}_{lim} = \frac{80\psi}{\sqrt{F_y}} \quad (\text{B.12})$$

Then, F_y is replaced by F_{cr} in equations B.8 and B.9;

$$F_{cr} = \left[1.677 - 0.677 \frac{w/t}{(w/t)_{lim}} \right] \quad \left(\frac{w}{t} \right)_{lim} \leq \frac{w}{t} \leq \frac{144\psi}{\sqrt{F_y}} \quad (\text{B.13})$$

$$F_{cr} = \frac{0.0332\pi^2 E}{(w/t)^2} \quad \frac{w}{t} > \frac{144\psi}{\sqrt{F_y}} \quad (\text{B.14})$$

Where $\psi = 2.62$ for F_y in *MPa* and $\psi = 1.0$ for F_y in *ksi*.

Appendix C

Angle iron member connections

Here one considers the connections of tower members. In South Africa the welding of power line towers is not permitted, owing to the high cost of welding and the additional cost that is involved of managing the quality of the welds. All tower members are bolted together. A review of tower bolting configuration is required, seeing that it is envisaged to apply similar principles when connecting CHS members.

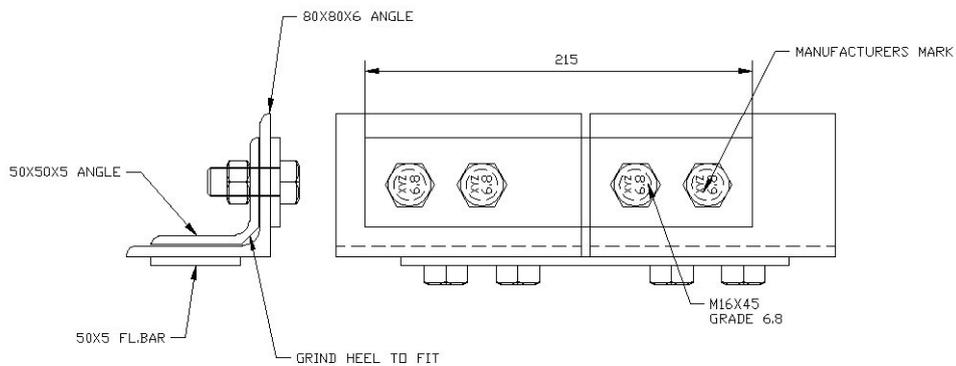


Figure C.1: Typical angle member connection detail.

Figure C.1 is an illustration of a typical connection. The typical fasteners used to assemble transmission towers are class 6.8 nuts and bolts. The ultimate tensile strength of these bolts are 600MPa and a yield strength of 480MPa . Although higher strength bolt grades are available, class 6.8 is used in order to minimize the risk of accidentally using the wrong bolt grade and thus jeopardizing the integrity of the connection strength.

The bolt shear capacity is calculated as follows (*ASCE 10-97:Design of latticed steel transmission structures 1997*):

$$F_v = 0.62F_u \quad (\text{C.1})$$

Where, F_u is the specified minimum tensile strength of the bolt material. The shear area is the gross cross-sectional area if the threads of the bolt are excluded from the shear plane.

The maximum bearing stress may be calculated as follows (*ASCE 10-97:Design of latticed steel transmission structures 1997*):

$$\frac{F_b}{dt} \leq 1.5F_u \quad (\text{C.2})$$

where

d = diameter of the bolt

t = thickness of the connected part

The following bolt spacing (e) is also required (*ASCE 10-97:Design of latticed steel transmission structures 1997*) in order to ensure the integrity of the connection:

C.0.3 End distance

For the end distance, the largest calculated value of the following three equations should be used:

$$e = 1.2P/F_u t \quad (\text{C.3})$$

$$e = 1.3d \quad (\text{C.4})$$

$$e = t + d/2 \quad (\text{C.5})$$

where

F_u = minimum specified tensile strength of the connected part

t = thickness of the connected part

d = nominal diameter of the bolt

P = force transmitted by the bolt

For redundant members, e_{min} shall be the larger of equations C.5 and C.6 as required by *ASCE 10-97: Design of latticed steel transmission structures* (1997):

$$e = 1.2d \quad (C.6)$$

C.0.4 Center-to-center spacing

This is the spacing (s_{min}) along the line of the transmitted force. The minimum distance shall not be less than:

$$s_{min} = 1.2P/F_u t + 0.6d \quad (C.7)$$

C.0.5 Edge distance

The minimum edge distance shall not be less than:

For a rolled edge:

$$f_{min} = 0.85e_{min} \quad (C.8)$$

For a sheared or mechanically guided flame cut edge:

$$f_{min} = 0.85e_{min} + 0.0625\psi \quad (C.9)$$

where $\psi = 25.4$ when f_{min} is in mm .

Appendix D

Ratio of I_{CHS}/I_{ANG} without Class 3 requirements

The solution that follow considers I_{CHS}/I_{ANG} without the requirements of a class 3 member: $a/t = 10.7$ and $D/t = 59.5$

Step 1: Expand and simplify moment of inertia equations for CHS:

$$I_{CHS} = \frac{\pi(R^4 - R_i^4)}{4} \quad (D.1)$$

where

R = outside radius

R_i = inside radius

Replace R_i with $(R-t)$

$$I_{CHS} = \frac{\pi(R^4 - (R-t)^4)}{4} \quad (D.2)$$

Expanded form:

$$I_{CHS} = \pi R^3 t - \frac{3}{2} \pi R^2 t^2 + \pi R t^3 - \frac{\pi t^4}{4} \quad (D.3)$$

Remove higher terms of (t)

$$I_{CHS} = \pi R^3 t \quad (D.4)$$

Step 2: Expand and simplify moment of inertia equations for angle section:

$$I_{ANG} = \frac{a^4 - b^4}{12} - \frac{0.5ta^2b^2}{a + b} \quad (D.5)$$

where a = angle section leg length

$$b = (a - t)$$

Expanded form:

$$I_{ANG} = -\frac{a^4t}{4a - 2t} + \frac{2a^3t^2}{4a - 2t} + \frac{a^3t}{3} - \frac{a^2t^3}{4a - 2t} - \frac{a^2t^2}{2} + \frac{at^3}{3} - \frac{t^4}{12} \quad (D.6)$$

Remove higher terms of (t)

$$I_{ANG} = -\frac{a^4t}{4a - 2t} + \frac{a^3t}{3} \quad (D.7)$$

Step 3: Set $A_{CHS} = A_{ANG}$ and solve R:

$$\pi(R^2 - (R - t)^2) = t(2a - t) \quad (D.8)$$

$$R = \frac{2a + (\pi - 1)t}{2\pi} \quad (D.9)$$

Step 4: Replace R in I_{CHS} / I_{ANG} :

$$K = \frac{3(2a - t)(2a + (\pi - 1)t)^3}{4\pi^2a^3(a - 2t)} \quad (D.10)$$

where $K = I_{CHS} / I_{ANG}$

Step 5: Solve for t = 0:

$$K = \frac{12}{\pi^2} \quad (D.11)$$

Appendix E

Ratio of I_{CHS}/I_{ANG} with Class 3 requirements

The analytical solution that follow includes the code requirements for class 3 structural members: $a/t = 10.7$ and $D/t = 59.5$. The simplification for the moment of inertia from appendix D will again be used here.

$$t_{ANG} = 0.09348a \quad (E.1)$$

$$t_{CHS} = 0.03361R \quad (E.2)$$

Step 1: Set $A_{CHS} = A_{ANG}$ and replace values of (t), Solve R:

$$\pi(R^2 - (R - t)^2) = t(2a - t) \quad (E.3)$$

$$\pi(R^2 - (R - (0.03361R))^2) = (0.09348a)(2a - (0.09348a)) \quad (E.4)$$

$$R = 0.92648a \quad (E.5)$$

Step 2: Solve $K = I_{CHS}/I_{ANG}$:

Replace R from equation E.5 into equation E.2 and replace (t) into I_{CHS}/I_{ANG} .

$$K = \frac{3(2a - t)(2a + (\pi - 1)t)^3}{4\pi^2 a^3 (a - 2t)} \quad (E.6)$$

*APPENDIX E. RATIO OF I_{CHS}/I_{ANG} WITH CLASS 3 REQUIREMENTS*100

which gives $K = 11.709$.

This means that the buckling resistance of a CHS member in compression will be 11.7 times more. The basic assumption that may be drawn from this is that longer unsupported tower members can be used in the power line tower. This in turn affects the load, mass and economics of the structure.

Appendix F

Hollo bolts capacities and installation details

Hot Dipped Galvanized Hollo-Bolt

Note: Three Lengths are available as indicated by the product code.

Bolt Size Metric (US Equiv)	Product Code	Clamping Range		Across Flats Main Body	Tightening Torque (ft lb)	Shank Diam. (A)	Hole Size (Z)	Structural Tube	Safe Working Load* (5:1 Factor of Safety)	
		Combined Material Thickness (W) Min	Max						Tensile (lb)	Shear (lbs)
M8 (5/16")	LHBM08 #1-HDG LHBM08 #2-HDG LHBM08 #3-HDG	1/8" 7/8" 1-5/8"	7/8" 1-5/8" 2-3/8"	3/4"	16	9/16"	9/16"	5" x 5" x 3/16", 1/4", or 5/16"	899	1124
M10 (3/8")	LHBM10 #1-HDG LHBM10 #2-HDG LHBM10 #3-HDG	1/8" 7/8" 1-5/8"	7/8" 1-5/8" 2-3/8"	15/16"	30	3/4"	3/4"	5" x 5" x 5/16", 3/8" or 1/2"	1910	2248
M12 (1/2")	LHBM12 #1-HDG LHBM12 #2-HDG LHBM12 #3-HDG	1/8" 1" 1-13/16"	1" 1-13/16" 2-3/4"	1-3/16"	59	13/16"	13/16"	5" x 5" x 5/16", 3/8" or 1/2"	2360	3372
M16 (5/8")	LHBM16 #1-HDG LHBM16 #2-HDG LHBM16 #3-HDG	5/16" ** 1-1/8" 2"	1-1/8" 2" 2-13/16"	1-3/8"	140	1-1/16"	1-1/8"	5" x 5" x 1/2"	4946	6744
M20 (3/4")	LHBM20 #1-HDG LHBM20 #2-HDG LHBM20 #3-HDG	5/16" ** 1-5/16" 2-3/8"	1-5/16" 2-3/8" 3-3/8"	1-13/16"	221	1-5/16"	1-3/8"	5" x 5" x 1/2"	8992	8992

Figure F.1: Hollo Bolt capacities.

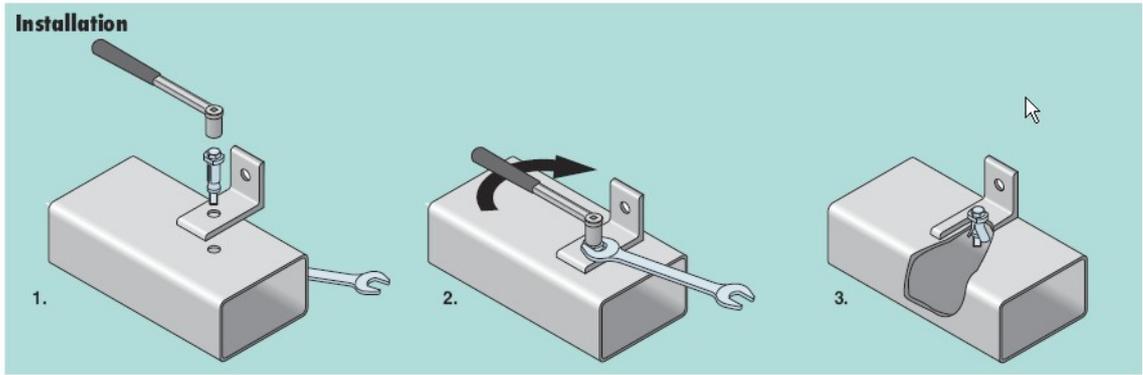


Figure F.2: Holo Bolt installation.

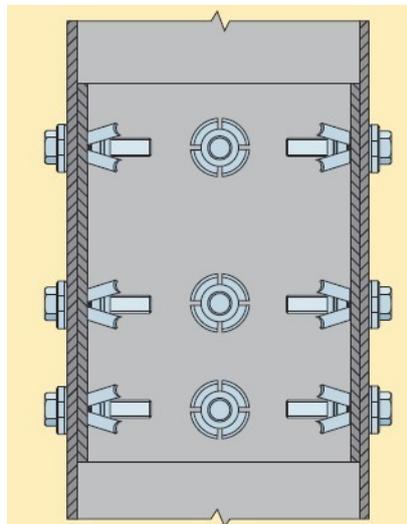


Figure F.3: Holo Bolt installation section view.

Appendix G

Huck bolt fasteners



Figure G.1: Huck BOM fastener.

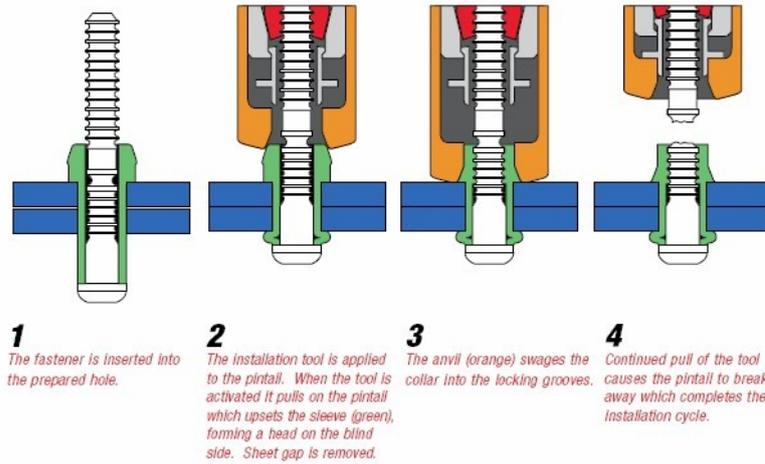


Figure G.2: Huck BOM fastener installation details.

Air Driven Tools



Model	246	256
Weight:	11.1 lbs	11.1 lbs
Length:	8.8"	7.8"
Height:	15.1"	14.9"
Width:	6.1"	6.1"

Figure G.3: Huck BOM fastener hand tool.

Appendix H

Design guidelines for truss design

The following guidelines have been suggested in order to achieve an optimized truss design (Wardenier 2001):

- (a) Select geometry that will minimize the number of joints. This will increase manufacturing productivity and reduce cost.
- (b) The depth of the truss panel can be selected as 1/10 to 1/16 the span length.
- (c) Members should be selected to provide sufficient joint strength and economical fabrication.
- (d) Designing members based on loads only could lead to over stiffening of joints.
- (e) Members can be designed based on pin jointed theory.
- (f) If there is sufficient rotation capacity in the joint, the secondary bending moments can be ignored.
- (g) For joint design and tension members, nodding eccentricities should be within the specified limits. See figure H.1.
- (h) Members in compression should be checked for bending effects developed by nodding eccentricities (Beam-Column design with moments distributed to the chord sections).
- (i) Based on fabrication, gap joints are preferred to partial overlap joints.

- (j) The minimum gap should be $g \geq t_1 + t_2$. See figure H.2.
- (k) The overlap in overlap joints should be a minimum $O_v \geq 25\%$.
- (l) In a typical truss, the chords in compression weights 50 % of material weight, 30 % for the cord in tension and the bracing members approximately 20%. Thus the chords in compression should be the members that are optimized for weight saving. The outer surface must be optimized to reduce paint area.
- (m) When chord diameter or width to thickness ratio decreases, the joint strength increases. Thus compression members will be a trade-off between joint strength and buckling strength (Stocky members). Low values of diameter or width to thickness ratio should be selected for tension members.
- (n) Increasing the chord to brace thickness (t_o/t_i) will increase the joint strength efficiency (joint strength divided by the brace yield load $A_i \cdot f_y$). In addition the weld for a thin walled brace is less than for thick walled brace with the same cross section.
- (o) If commercially available, use higher strength steels for chord members to increase the joint strength.
- (p) Approach multiplanar joints in a similar way as uniplanar joints. The depth of multiplanar joints will vary between 1/15 to 1/18 l .

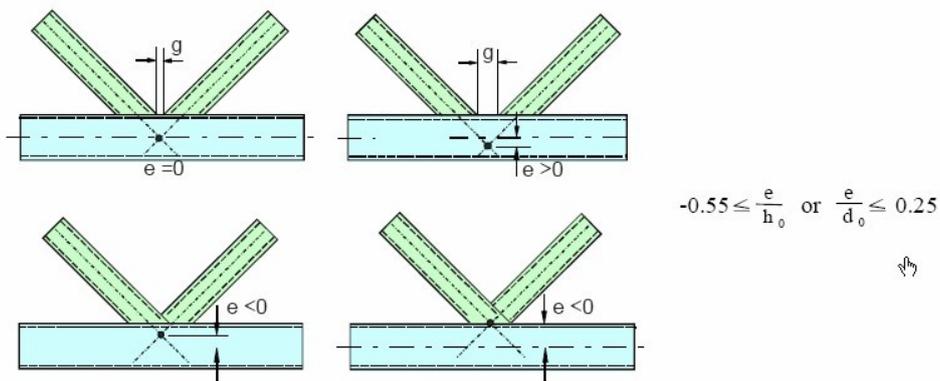


Figure H.1: Noding eccentricity limits (Wardenier 2001).

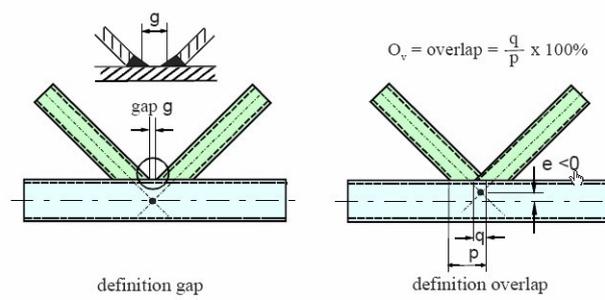


Figure H.2: Illustration of gap and overlap (Wardenier 2001).

Appendix I

Failure modes in circular hollow section connections

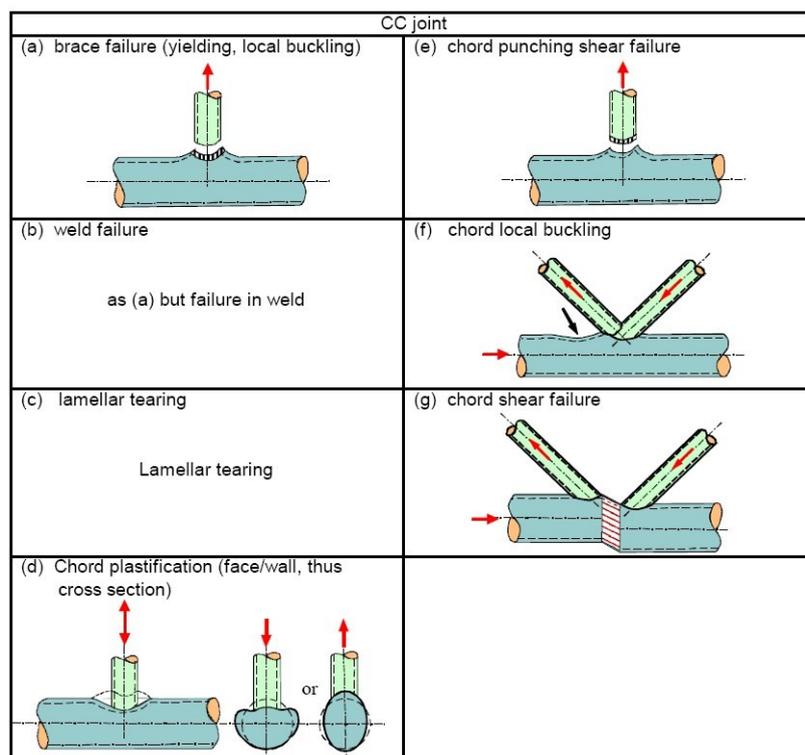


Figure I.1: Typical CHS failure modes (Wardenier 2001).

Appendix J

Welded CHS connection capacities

d_1/t_1 limits for which the joint efficiencies derived from Figs. 8.17 to 8.20 can always be used		efficiency limit * for compression brace					
		f_{y1}	d_1/t_1				
yield stress	d_1/t_1 limit			30	35	40	45
$f_y = 235 \text{ N/mm}^2$	$d_1/t_1 \leq 43$	235	1	1	1	1	0.9
$f_y = 275 \text{ N/mm}^2$	$d_1/t_1 \leq 37$	275	1	1	1	0.9	0.9
$f_y = 355 \text{ N/mm}^2$	$d_1/t_1 \leq 28$	355	1	0.9	0.9	0.8	0.8

Figure J.1: d_1/t_1 limits for compression bracing members (Wardenier 2001).

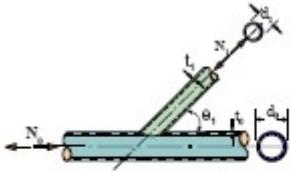
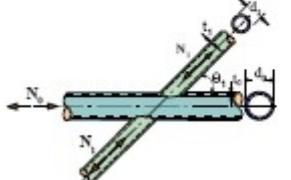
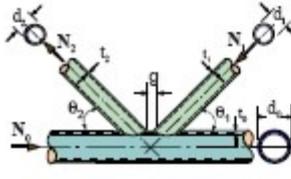
Type of joint	Design strength (i = 1, 2)			
T- and Y-joints	chord plastification			
	$N_1^* = \frac{f_{y0} \cdot t_0^2}{\sin \theta_1} \cdot (2.8 + 14.2 \beta^2) \cdot \gamma^{0.2} \cdot f(n')$ <p style="text-align: right;">(eq. 4.2.1)</p>			
X-joints	chord plastification			
	$N_1^* = \frac{f_{y0} \cdot t_0^2}{\sin \theta_1} \cdot \left[\frac{5.2}{1 - 0.81 \beta} \right] \cdot f(n')$			
K and N gap or overlap joints	chord plastification			
	$N_1^* = \frac{f_{y0} \cdot t_0^2}{\sin \theta_1} \cdot \left(1.8 + 10.2 \frac{d_1}{d_0} \right) \cdot f(\gamma, g') \cdot f(n')$ $N_2^* = N_1^* \cdot \frac{\sin \theta_1}{\sin \theta_2}$			
general	punching shear			
punching shear check for T,Y,X and K,N,KT joints with gap	$N_1^* = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot d_1 \cdot \frac{1 + \sin \theta_1}{2 \sin^2 \theta_1}$			
functions				
$f(n') = 1.0 \text{ for } n' \geq 0 \quad n' = \frac{f_{op}}{f_{y0}}$ <p style="text-align: center;">(tension)</p> $f(n') = 1 + 0.3 n' - 0.3 n'^2 \text{ for } n' < 0$ <p style="text-align: center;">(compression)</p>	$f(\gamma, g') = \gamma^{0.2} \cdot \left[1 + \frac{0.024 \gamma^{1.2}}{\exp(0.5 g' - 1.33) + 1} \right]$			
validity ranges				
$0.2 < \frac{d_1}{d_0} \leq 1.0$	$\frac{d_1}{2t_1} \leq 25$ and see table 8.1a	$30^\circ \leq \theta_1 \leq 90^\circ$ $-0.55 \leq \frac{e}{d_0} < 0.25$	$\gamma \leq 25$ $\gamma \leq 20$ (X-joints)	$Ov \geq 25\%$ $g \geq t_1 + t_2$

Figure J.2: Load capacities of welded CHS joints (Wardenier 2001).

Appendix K

Application of SANS 10162-1

The next set of equations shows the implementation of the National Code for structural member design (structural use of steel Part 1:Limit-state design of hot-rolled steelwork 2005). It is not recommended to deviate from this design code. Where other design codes are used for designing members in compression or combined compression and bending, it should clearly be benchmarked to this design code and its application should be discussed with other professional structural engineers and tower designers.

The calculations that follow are also typically what Prokon produces. This will allow the design engineer to evaluate the design of each member.

K.1 Slenderness

From Table 4 (structural use of steel Part 1:Limit-state design of hot-rolled steelwork 2005):

$$\begin{aligned} D_{Tratio} &= \frac{D}{T} f_y && \text{(K.1)} \\ &= \frac{152}{6.0} 355 \\ &= 8993.333 \end{aligned}$$

$$D_{Tratio} < 13000$$

Thus the member is a Class 1 section.

K.2 Maximum slenderness ratio (Sec 10.4.2.1):

$$\begin{aligned}
 X_{ratio} &= \frac{K_x L_x}{r_x} & (K.2) \\
 &= \frac{(1.0)(2585)}{52} \\
 &= 49.712
 \end{aligned}$$

The slenderness ratio is equal for both the x and y axis. The maximum slenderness ratio for members in compression is 200. The slenderness of this member is less than 200, this member is OK.

K.3 Cross-sectional strength (Sec 13.8.3a):

$$\begin{aligned}
 C_r &= \frac{\phi A f_y}{1000} & (K.3) \\
 &= \frac{(0.9)(2750.0)(355)}{1000} \\
 &= 878.625 \text{ kN}
 \end{aligned}$$

$$\begin{aligned}
 M_r &= \frac{\phi Z_p f_y}{1 \times 10^6} & (K.4) \\
 &= \frac{(0.9)(128000)(355)}{1 \times 10^6} \\
 &= 40.896 \text{ kNm}
 \end{aligned}$$

End moment factors (Section 13.8.5):

$$\begin{aligned}
 \kappa_x &= \frac{M_{Ux-min}}{M_{Ux-max}} & (K.5) \\
 &= \frac{2.11}{-3.40} \\
 &= -0.6206
 \end{aligned}$$

$$\begin{aligned}
 \kappa_y &= \frac{M_{Uy-min}}{M_{Uy-max}} & (K.6) \\
 &= \frac{-14.13}{-17.00} \\
 &= 0.8312
 \end{aligned}$$

$$\begin{aligned}
 \omega_x &= 0.6 - 0.4\kappa_x & (K.7) \\
 &= 0.848
 \end{aligned}$$

$$\begin{aligned}
 \omega_y &= 0.6 - 0.4\kappa_y & (K.8) \\
 &= 0.0268
 \end{aligned}$$

$$\omega = \max(\omega_y, 0.4) = 0.4$$

$$\begin{aligned}
 C_{ex} = C_{ey} &= \frac{\pi^2 EI_x}{1000(K_x L_x)^2} & (K.9) \\
 &= 2171181 kN
 \end{aligned}$$

$$\begin{aligned}
 U1_x &= \frac{\omega_x}{1 - \frac{C_u}{C_{ex}}} & (K.10) \\
 &= 0.9595
 \end{aligned}$$

$$\begin{aligned}
 U1_y &= \frac{\omega_y}{1 - \frac{C_u}{C_{ey}}} & (K.11) \\
 &= 0.4526
 \end{aligned}$$

$U1 = 1.0$ for members in braced frames.

$$\frac{C_u}{C_r} + \frac{U1_x M_{ux}}{M_{rx}} + \frac{U1_y M_{uy}}{M_{ry}} \leq 1.0 = 0.7858(OK)$$

K.4 Overall member strength (Sec 13.8.3b):

$$\begin{aligned} f_{ex} = f_{ey} &= \frac{\pi^2 E}{\left[\frac{KL}{r}\right]^2} \\ &= 789.57 MPa \end{aligned} \quad (K.12)$$

$$\begin{aligned} f_{ez} &= \frac{\frac{\pi^2 EC_w}{(K_z L_z)^2} + GJ}{Ar_o^2} \\ &= 77000 MPa \end{aligned} \quad (K.13)$$

$C_w = 0$ for circular hollow sections.

$$f_e = \min(f_{ex}, f_{ey}, f_{ez}) = 789.6 \text{ MPa}$$

$$\begin{aligned} \lambda &= \sqrt{\frac{f_y}{f_e}} \\ &= \sqrt{\frac{355}{789.6}} \\ &= 0.6705 \end{aligned} \quad (K.14)$$

$$\begin{aligned} C_r &= \frac{\phi A f_y [1 + \lambda^{2n}]^{-\frac{1}{n}}}{1000} \\ &= 705.226 kN \end{aligned} \quad (K.15)$$

$$\frac{C_u}{C_r} + \frac{U1_x Mux}{M_{rx}} + \frac{U1_y Muy}{M_{ry}} \leq 1.0 = 0.6257(OK)$$

K.5 Lateral torsional buckling strength (13.8.3c):

$$\begin{aligned} M_r &= \frac{\phi f_y Z_p l x}{1x10^6} \\ &= \frac{(0.9)(355)(128000)}{1x10^6} \\ &= 40.896kNm \end{aligned} \quad (K.16)$$

C_r based on weak axis bending

$$\begin{aligned} \lambda_y &= \frac{KyLy}{r_y} \sqrt{\frac{fy}{\pi^2 E}} \\ &= 0.6705 \end{aligned} \quad (K.17)$$

$$\begin{aligned} C_r &= \frac{\phi A f_y [1 + \lambda^{2n}]^{-\frac{1}{n}}}{1000} \\ &= 704.958kN \end{aligned} \quad (K.18)$$

$$\frac{C_u}{C_r} + \frac{U1_x Mux}{M_{rx}} + \frac{U1_y Muy}{M_{ry}} \leq 1.0 = 0.629(OK)$$

The design of of all tubular members on power line towers for South African conditions must follow the approach as outlined above unless the cross-sectional stability and overall strength can be proven otherwise. All the members in the test tower has been designed following this approach.

Appendix L

Bolted end plate connections

This section discusses in detail the design considerations for bolted end plate connections. These type of connections will be used throughout a typical transmission tower for structural and non-structural members. Figures 6.10 and 6.11 are shown here again for ease of reading.

First considering the T-stub connection (figure 6.10). A load of 220 kN is transferred through this member. The shear resistance of a M16 class 8.8 bolt in single shear is $54\text{ kN} \times 1.43$ (thread not in shear plane) = 77 kN . It can be seen that a minimum of 3 bolts are required. The author selected to use 4 bolts. When designing structures for commercial use, a more economical approach will be used and 3 bolts would have been selected.

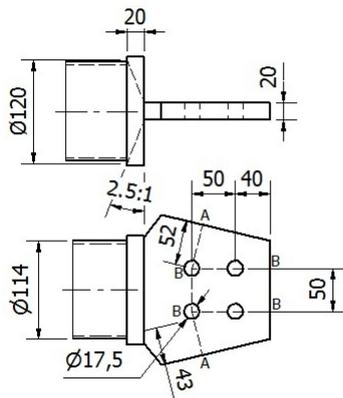


Figure L.1: T-stub welded attachment.

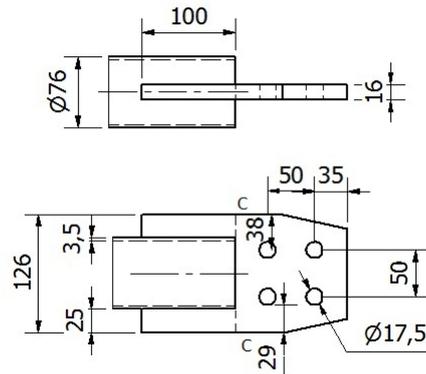


Figure L.2: Slotted end plate welded attachment.

Following the load path, two failure lines in the plate is most likely to

occur, A-A and B-B. The design approach is to determine the net area of the plate and then determine the tensile resistance of the member (Clause 13.2 SANS 10162: Part 1):

$$\begin{aligned} A_{ne} &= (w - 2d')t & (L.1) \\ &= [(52 + 50 + 52) - (2 \times 17.5)]20 \\ &= 2380 \text{mm}^2 \end{aligned}$$

The tensile resistance (T_r) is then given by:

$$\begin{aligned} T_r &= 0.85\phi A_{ne} f_u & (L.2) \\ &= 0.85(0.9)(2380)(480) \\ &= 874 \text{kN} \end{aligned}$$

The effect of shear can also be considered which will then yield 874 kN multiply 0.6 = 524 kN.

The fracture on line B-B is a combination of tension and shear block failure (Clause 13.11 SANS 10162: Part 1):

Net area in tension:

$$\begin{aligned} A_{nt} &= (c - d')t = (50 - 17.5)20 \\ &= 650 \text{mm}^2 \end{aligned}$$

Gross area in shear:

$$\begin{aligned} A_{gv} &= (2a)t = 2(50 + 40)20 \\ &= 3600 \text{mm}^2 \end{aligned}$$

Net area in shear is calculated by subtracting the area of the bolts:

Gross area in shear:

$$\begin{aligned} A_{nv} &= A_{gv} - (2 \cdot 1.5 \cdot 17.5 \cdot 20) & (L.3) \\ &= 3600 - 1050 \\ &= 2550 \text{mm}^2 \end{aligned}$$

The tensile resistance (T_r) is the lesser of:

$$\begin{aligned}
 T_r &= \phi A_{nt} f_u + 0.6 \phi A_{gv} f_y & (L.4) \\
 &= [(0.9)(650)(480)] + [(0.6)(0.9)(3600)(300)] \\
 &= 864kN
 \end{aligned}$$

$$\begin{aligned}
 T_r &= \phi A_{nt} f_u + 0.6 \phi A_{nv} f_u & (L.5) \\
 &= [(0.9)(650)(480)] + [(0.6)(0.9)(2550)(480)] \\
 &= 661kN
 \end{aligned}$$

The factored bearing resistance (B_r) of the bolted connection is the lesser of (Clause 13.10 SANS 10162: Part 1) :

$$\begin{aligned}
 B_r &= 3\phi_{br} t d n f_u & (L.6) \\
 &= 3(0.67)(20)(16)(4)(480) \\
 &= 1235kN
 \end{aligned}$$

$$\begin{aligned}
 B_r &= \phi_{br} a t n f_u & (L.7) \\
 &= (0.67)(40)(20)(4)(480) \\
 &= 1029kN
 \end{aligned}$$

It can thus be seen that the factored resistance (T_r and B_r) of the connection exceeds the ultimate load (T_u) of the member.

Next is the weld that joins the flat plate to the circular end plate. If we use the recommended weld design approach (Clause 13.13 SANS 10162: Part 1), the shear resistance in the weld is $220kN/114\text{ mm} = 1.93\text{ kN/mm}$ and again dividing that in half for each side of the plate gives 0.96 kN/mm required to resist the ultimate load. From table 7.6 (*Southern African Steel Construction Handbook* 2008), a 5 mm weld will be adequate. The weld size can be increased to say 8 mm in order to account for any secondary effects in the connection.

Next, the load dispersion should be considered in order to account for any shear lag effect (Figure L.3). Packer et al (Packer & Henderson 1997) suggest that the load dispersion is equal to $5t_p + b_p$, where t_p is the thickness

of the end plate and b_p is the thickness of the Tee plate. It is suggested by the author to always select plates that will ensure that the full cross-section of the tube is effective in load transfer.

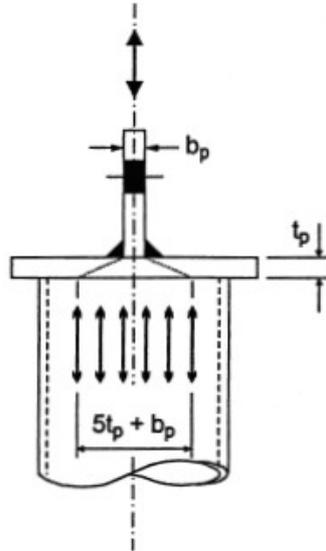


Figure L.3: Load dispersion through T-stub connection.

In the current connection, both the end plate and Tee plate is 20 mm thick, this yields a dispersion width of 120 mm. This is more than the tube diameter of 114 mm. The full cross-section of the tube is thus effective in load transfer.

The perimeter length of the tube is 358 mm. The shear load through the weld is thus 0.614 kN/mm. A minimum weld size of 5 mm can be used, although it is recommended to increase the weld size to say 8 mm in order to account for secondary effects.

The second brace end connection (figure 6.11) has a similar approach to the bolted connection and will not be considered again. The only additional fracture line in the plate that should be considered is on failure line C-C. Clause 12.3.3.3 SANS 10162: Part 1 indicates that the effective net area is dependant on the weld length (L) and member width (w):

$$\begin{aligned}
 2w > L \geq w & \quad (L.8) \\
 A_{ne2} = 0.5wt + 0.25Lt &
 \end{aligned}$$

A_{ne2} is then calculated to be 1008mm^2 . Using a shear lag factor of 0.75 for the plate beyond the weld, the area is $0.75(126-76)16 = 600\text{mm}^2$. The total load resisting area is $1008 + 600 = 1608\text{mm}^2$. The tensile resistance is then:

$$\begin{aligned}
 T_r &= 0.85\phi A'_{ne} f_u & (L.9) \\
 &= 0.85(0.9)(1608)(480) \\
 &= 590.5\text{kN}
 \end{aligned}$$

The load in this connection is 200 kN . The length of one weld is 100 mm . The required shear resistance of the weld is 0.5 kN/mm . A weld size of 5 mm will be adequate.

The resistance of the parent metal (tube) is given by Clause 13.13.2.2 SANS 10162: Part 1 as:

$$\begin{aligned}
 V_r &= 0.67\phi_w A_m f_u & (L.10) \\
 &= 0.67(0.67)(4 \cdot 100 \cdot 3.5)(480) \\
 &= 301.6\text{kN}
 \end{aligned}$$

It has also been suggested that shear lag effects can be neglected if ($L \geq 1.2d$) and in some cases 1.3. In this connection, the weld length (L) is 1.3d, thus the section is fully effective. It is suggested that 1.3d should be used for power line structures.

Appendix M

Design of gusset plate connections

M.1 Resistance in chord plastification, $N_1 Rd_{PL}$:

$$N_1 Rd(PL) = 5(1 + 0.25\eta)f(n')(f_{yo})(to^2) \quad (M.1)$$

where:

$$\eta = \frac{B}{D} = \frac{382}{140} = 2.72 \quad (M.2)$$

With an upper limit for (η) is: $\eta \leq 4$.

$$(1 + 0.25\eta) = 1 + 0.25(2.72) = 1.68 \quad (M.3)$$

There is a reduction in the strength of the connection when the chord is in compression, this reduction factor will be calculated next. The increase in stiffness when the chord is in tension will be ignored. Before the calculations are further discussed, the consideration of the loading for calculating the connection strength must be evaluated. In the design of power line towers, there are many different types of load cases and in a full tower there are many connections with this type of layout. The author suggest that the algebraic sum of the brace loads for the various load cases must be calculated and the associated load case should then be used throughout for load selection in braces and chord member.

Also instead of calculating n' from equation M.4, the member usage that was calculated in the structural software for overall member strength must

be used. This usage also includes the effect of combined loading (axial and bending loads). This value should then be used in equation M.5

$$n' = \frac{N_{op}}{A_o \cdot f_{yo}} \quad (M.4)$$

In this case, the maximum usage from load case (LC7) was 0.60.

$$f(n') = 1 + 0.3(n') - 0.3(n')^2 = 0.712 \quad (M.5)$$

f_{yo} is the yield strength of the chord member (355 MPa) and t_o is the thickness of the chord member (4 mm). Taking all of this into consideration yields a chord plastication resistance of:

$$N_1 R_{d_{PL}} = 33.97 kN \quad (M.6)$$

M.2 In-plane moment resistance, ($M_{ip.Rd}$)

The moment resistance ($M_{ip.Rd}$) can now be calculated from the chord plastication resistance:

$$\begin{aligned} M_{ip.Rd} &= (N_1 R_{d_{PL}})(B) \\ &= (33.97 kN)(0.382 m) \\ &= 12.97 kN.m \end{aligned} \quad (M.7)$$

M.3 In plane bending moment, (M_{ip})

The in-plane bending moment is directly related to the brace loading. The brace load component parallel with the chord creates a bending moment that is proportional with the load eccentricity.

The load parallel to the chord, N_p :

$$\begin{aligned}
 Np &= N1(\text{Cos}\theta1) + N2(\text{Cos}\theta2) \\
 &= 87.52 + 90.44 \\
 &= 177.96kN
 \end{aligned}
 \tag{M.8}$$

In-plane bending moment, M_{ip} :

$$\begin{aligned}
 M_{ip} &= (Np)\left(\frac{Do}{2}\right) \\
 &= (177.96)\left(\frac{140}{2}\right) \\
 &= 12.45kN.m
 \end{aligned}
 \tag{M.9}$$

Considering the additional effect of multiplanar connections (0.9) the resistance to chord plastification is less than the in-plane moment. It is decided that the approach suggested by the author for selecting load combinations will prove to be conservative.

M.4 Resistance in punching shear, $N_1 Rd_{Pu}$

Packer & Henderson (1997) gives the following equation for chord punching resistance:

$$\begin{aligned}
 N_1 Rd_{Pu} &= 1.16(Fyo)(to) \\
 &= 1.16(355MPa)(4mm) \\
 &= 1.65kN/mm
 \end{aligned}
 \tag{M.10}$$

M.5 Punching force, N_{pu}

The punching force (N_{pu}) on the chord is a combined load of axial and bending stresses expressed as load per mm of gusset plate. The punching force can be calculated from equation M.11 and M.13.

$$fa = \frac{5057.6}{(4)(382)} = 3.31N/mm^2 \quad (M.11)$$

$$\begin{aligned} S &= \frac{bd^2}{6} \\ &= \frac{(0.016)(0.382)^2}{6} \\ &= 0.389E - 3m^3 \end{aligned} \quad (M.12)$$

$$\begin{aligned} fb &= \frac{M}{S^1} \\ &= \frac{12450}{1.389E - 3} \\ &= 31.99N/mm^2 \end{aligned} \quad (M.13)$$

Thus,

$$(fa + fb)t = (3.31 + 31.99)4 = 141.2N/mm = 0.141kN/mm \quad (M.14)$$

The punching force per *mm* is less than the resistance to punching shear, thus connections OK.

The final equation that will be analysed here is the I-beam to main leg connection. Again, to select the worst loading on the connection, the maximum load in the cross arm member that connects with this connection was selected as the worst case. The loads in the main leg member associated with that load case was then used as the loads in the leg member:

M.6 Punching Force, N_{Pu}

$$\left[\frac{N1}{A1} + \frac{M_{f1}}{S1} + \frac{N_{f2}}{S2} \right] t1 = N_{Pu} \quad (M.15)$$

where,

¹S is the elastic modulus of the gusset plate

$$\frac{N1}{A1} = \frac{275000}{3120} = 88.14MPa \quad (M.16)$$

$$\frac{M_{f1}}{S1} = \frac{2360000}{288480} = 8.18MPa \quad (M.17)$$

$$\frac{M_{f2}}{S2} = \frac{3250000}{67600} = 48.1MPa \quad (M.18)$$

$$N_{Pu} = (88.14 + 8.18 + 48.1)12 = 1.73kN/mm$$

$N1$ = Axial force in I-beam connection

$A1$ = Area of I-beam flanges

M_{f1} = Bending moment in x-x

M_{f2} = Bending moment in y-y

$S1$ = Elastic modulus x-x

$S2$ = Elastic modulus y-y

M.7 Resistance in punching shear, N_1Rd_{Pu}

$$\begin{aligned} N_1Rd_{Pu} &= 1.16(Fyo)(to) & (M.19) \\ &= 1.16(355MPa)(6mm) \\ &= 2.43kN/mm \end{aligned}$$

M.8 Resistance in chord plastification, N_1Rd_{PL} :

$$N_1Rd(PL) = \left(\frac{5.0}{1 - 0.81\beta}\right)(1 + 0.25\eta)f(n')(fyo)(to^2) \quad (M.20)$$

where:

$$\beta = \frac{b1}{do} = \frac{130}{140} = 0.93 \quad (M.21)$$

$$\eta = \frac{h1}{do} = \frac{208}{140} = 1.49 \quad (M.22)$$

In this case, the maximum usage from load case (LC3) was 0.35.

$$f(n') = 1 + 0.3(n') - 0.3(n')^2 = 0.858 \quad (\text{M.23})$$

$$f_{yo}t_o^2 = (350)(6)^2 = 12600 \quad (\text{M.24})$$

$$\left[\frac{5.0}{1 - 0.81\beta} \right] = 20.267 \quad (\text{M.25})$$

$$(1 + 0.25\eta) = 1 + 0.25(1.49) = 1.37 \quad (\text{M.26})$$

$$N1Rd(PL)_{XP-4} = (20.267)(1.37)(0.858)(12600) = 300.17kN$$

$$N1Rd(PL)_{XP-1} = (20.267)(1.0)(0.858)(12600) = 219.102kN$$

M.9 In-plane bending strength

$$M_{ip} = h1N_{XP-1} = (0.208)(219.102) = 45.57kNm \quad (\text{M.27})$$

M.10 Out-plane bending strength

$$M_{op} = 0.5b1N_{XP-4} = 0.5(0.130)(300.17) = 19.51kNm \quad (\text{M.28})$$

It can be seen from the above equations that the connection has adequate strength to take the applied load.

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