A Comparison of Australian and American Design of Double Angle Connections

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A large boiler support structure for a coal-fired power plant in Australia was designed by a structural engineer in the United States. Approximately 9,000 tons (8,181 tonnes) of structural steel for this project was supplied from a steel fabricator in China, using shop drawings prepared by an American detailing firm. A registered professional engineer from the designer's Australian offices supervised the work in the United States to ensure compliance with local codes and standards. Member sizes were selected from ASTM shapes, using the provisions of the Australian Standard for steel structures AS4100 (Council of Standards, 1998). However, to expedite steel delivery, it was decided to allow the detailing firm to prepare connection designs using the American Institute for Steel Construction's (AISC), *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design* (ASD) (AISC, 1989). This situation presented two unique challenges for the project engineers: 1) the structure is designed using an ultimate strength code while the connection designs use the allowable stress methods and 2) it was necessary to demonstrate that United States connection designs are comparable to those produced using Australian standards. Demonstrating the compatibility of connection designs prepared using these two methods highlighted the unique difference between simple code compliance, and the art of connection design.

CODE COMPARISON

The first step in evaluating the potential for differences in connection design was to perform a comparison of the United States and Australian specifications. Appendix A tabulates those sections of each country's specifications that govern connection design. This appendix includes both American Load and Resistance Factor Design (LRFD) (AISC, 2001) and ASD. Numerical examples for bolted connections using ¾-in. diameter bolts are provided where practical to facilitate the comparison.

In general the code comparison showed that connections designed by Australian and United States' specifications address similar limit states. The Australian code does not explicitly require a check for block shear. This limit state is discussed in the Australian Institute of Steel Construction (AuISC) publication *Design of Structural Connections* (Hogan and Thomas, 1994). Section 5.17.3 of that publication describes the various aspects of block shear failures, and provides the limit state equations found in LRFD.

Design capacities for each limit state determined for the two countries' specifications compare favorably, with one obvious exception. The allowable capacity for a bearing bolt load in shear was as much as 20 percent greater per Australian specifications. Since it was decided that field-bolted connections should be used wherever possible on this job, this appeared to be a tremendous advantage to the design engineer. Most of the connections in the boiler support structure are gravity-loaded beam connections. Design of this type of connection is usually controlled by the shear capacity of bolts in bearing. In addition, the number of bolts to be installed significantly affects field labor. The potential for reducing the number of field bolts by as much as 20 percent was very attractive.

There is a distinct difference between simple code compliance and the art of structural design. This is true for any structure from the tallest high-rise building to the simplest of steel connections. An in-depth study of clip angle (angle cleats, in Australian steel jargon) design methods developed from an Australian point of view, compared with an American approach, provides a classic example of this difference.

BACKGROUND FOR COMPARISON OF DESIGN METHODS

"Designing" a clip angle to meet the AISC Specifications is a very simple proposition. Assuming the application is consistent with the assumptions built into the design manual, the engineer simply looks up the controlling capacity in Table 10-1 of Part 10 in LRFD. Similarly the AuISC provides design guidance and example problems for clip angles in *Design of Structural Connections* (Hogan and Thomas, 1994). When comparing the standard connection designs tabulated in these two publications it is expected that the Australian values will be greater, due to the higher bolt shears allowed by that code. In fact, exactly the opposite is true. In order to understand the difference, it is necessary to go behind the tabulated design values.

For the purpose of comparison, the connection used for the Australian Design Example 4.3.3 given in *Design of*

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Table 1. Metric—Imperial Comparison of "Standard" Connections		
	American	Australian
Angle Thickness	¼ in. (6.4 mm)	6.0 mm
Bolt Spacing	3 in. (75mm)	70 mm
Bolt Diameter	¾ in. (19.1 mm)	20 mm
F_y Angle	250 MPa	260 MPa
F_u Angle	400 MPa	410 MPa
F_y Beam	345 MPa	260 MPa
F_u Beam	450 MPa	410 MPa

Structural Connections, will be used. This example provides an extremely detailed check of a 410 UB 53.7 (W16×36) framing into a column flange as shown in Figure 1. The connection uses M20 (3/4-in.) bolts at a spacing of 70 mm (approximately 3 in.) in a pair of L 100 × 100 × 6 (L 4 × 4 × ¼). Numbers given in parentheses are nominal values for equivalent U.S. parts, based on a "soft" metric conversion. The design uses bearing bolts in standard holes, with threads included in the shear planes. This comparison will concentrate on the angle to beam portion of the connection, as the angle to column portion does not control the design.

The third edition of the LRFD Manual is used to derive the American connection capacity. In addition to the significant difference in the strength of wide flange members (F_y = 345 MPa for U.S. shapes versus F_y = 260 MPa for Australian shapes), there are some nuances in the comparison of an American design, whose foundation is in the world of imperial units, to a true "hard" metric design as performed in Australia. Specific imperial to metric conversions, and material differences for consideration are given in Table 1. It should be noted that certain sections of AISC's third edition of the *LRFD Specification* (AISC, 2001) use true metric bolt diameters (Table J3.1M for example). For the example being considered, the manual gives values for 20 mm diameter bolts which do not precisely compare with values for ¾-in. (19.05 mm) diameter.

A SIMPLE COMPARISON

To perform a very simple comparison of design methods, take the strength capacity from Table 10-1 of LRFD and that from the Australian example 4.4.3. Using the U.S. approach, the first check in Table 10-1 is the bolt and angle strength, which is given as 104 kips (462 kN) for a ¼-in. (6.4 mm) thick angle and four rows of ¾-in. diameter (20 mm) bolts. A review of this table shows that for bearing bolts this strength is controlled by the yielding of the angle. Bearing of the bolts on the web controls the strength of the beam web portion of this connection. The capacity of this limit state is found by multiplying the tabulated uncoped capacity per unit thickness 351 kips/in. (61.4 kN/mm), adjusted for the Australian beam strength (410 MPa / 450 MPa), by the beam web thickness (7.6 mm), to get a strength of 426 kN (96 kips). Thus the U.S. design is controlled by bolt bearing on the beam web.

The Australian approach is given in great detail in Design Example 4.4.3 of Hogan and Thomas (1994). This example shows five different general categories of global shear capacity checks (V_a through V_e). The limit states checked include; bolts to column, bolts to beam, clip angle shear, clip angle bending, and beam web shear. According to the Australian design example the connection is controlled by the bolts connecting to the beams web, with a limiting capacity of 352 kN (79 kips).

A comparison of Australian and American bolt and connection capacities given in the two country's design manuals is shown in Table 2. From this Table it can be seen that the Australian code allows a bolt capacity roughly 20 percent greater than the American code, yet the capacity of the "standard" American connection is about 25 percent stronger than the Australian. This drastic difference in connection strengths is worthy of a more detailed study. The simple comparison given above was performed using relatively similar connection dimensions. However, the minor

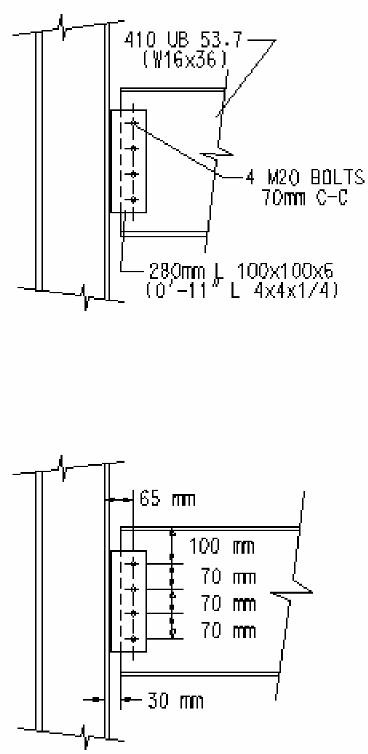

Fig. 1. Connection geometry.

Table 2. Comparison of Tabulated Connection Designs			
	American	**Australian**	**American/Australian**
M20 Double Shear	35 kips* (156 kN)	41 kips (185 kN)	84%
Connection Strength	96 kips (426 kN)	79 kips (352 kN)	122%

* Area of ¾-in. bolt = 0.442 in.^{2} Area of M20 bolt = 0.487 in.^{2}

From LRFD Table 7-10, Double Shear for ¾-in. bolt = 31.8 kips

∴ Double Shear for M20 = 31.8 kips (0.487 / 0.442) = 35 kips

dimensional difference as noted in Table 1 will be accounted for in the detailed comparison given below. For comparison purposes the dimensions given in the Australian design example will be used.

DETAILED COMPARISON

American practice for designing clip angles that use a single vertical row of bolts considers five different limit states when checking the angle to beam portion of the connection. The four primary limit states are: bolt shear, bolt bearing on the angles, shear yielding of the angles, and shear rupture of the angles. In addition, for uncoped beams a check of bolt bearing on the beam web is also considered. If the top flange of the beam is coped then a block shear check of the web is required. For beams that are coped on both the top and bottom flanges, limit states of shear yielding and shear rupture of the beam web would also be included. Appendix B gives detailed calculation of the American standard practice limit states for the connection shown in Figure 1. This Appendix also includes a check of shear yielding for the beam web. Because the beam is uncoped, this might be thought of as a check of the structure rather than the connection. The strength capacities for the American limit states are given in Table 3.

Australian practice for designing double clip angles that use a single column of bolts considers nine limit states for this connection plus a check of the shear yielding of the beam web. These nine limit states are; bolt shear, bolt bearing on the angle, bolt bearing on the beam web, shear rupture vertically on the angle, shear rupture vertically on the beam web, shear rupture horizontally on the angle, shear rupture horizontally on the beam web, shear yielding of the angle, and bending of the angle. Additional checks are required for beams that are coped. The strength capacities determined using Australian practice are given in Table 4. Detailed calculations for these values may be found in Hogan and Thomas (1994) and are summarized in Appendix C. Strength capacity for shear rupture horizontally on the angle, and shear rupture vertically on the beam web, are not included in Table 4, because total connection strength for these limit states is not provided in the reference. Bending of the angle is not included in this table, because it only affects the outstanding leg and thus is not related to the capacity of the angle to beam portion of the connection.

DIFFERENCES IN DESIGN APPROACH

The difference (55 kN, 12 kips) between the capacity of a clip angle connection, designed using American standard practice (407 kN, 91 kips) and that following an Australian approach (352 kN, 79 kips), is not the result of specified strength limitation in the steel design codes of the two countries. Instead, it is more an interpretation of the connection behavior by defining the various limit states that need to be investigated. Figure 2 shows the two controlling limit states, horizontal beam web tear out, and shear rupture of the connecting angles, based on the Australian and American methods respectively.

Horizontal beam web tear out, as a limit state for a gravity-loaded beam is highly unlikely. In order for this mechanism to occur a number of conditions would have to apply:

1. The angle legs connecting to the column must be totally inflexible.
2. The connection of the angle to the column must be totally rigid, that is, the heel of the angle must remain in solid contact with the column for its entire length.
3. The column must not yield in any manner, neither by gradual bending of the story length nor by localized yielding of the column flanges or web.
4. The end of the beam must experience a large rotation to the extent that web yielding will eventually lead to rupture.
5. The beam web must be allowed to slip relative to the encompassing angles as the beam end rotates.

Although each of these conditions is theoretically possible to attain in a laboratory it is unlikely that they will occur

Table 3. American Limits States Using Australian Geometry and Material	
Limit State	**Strength Capacity**
Bolt shear Bolt bearing on angles	98 kip (438 kN) 159 kip (708 kN)
Bolt bearing on beam web Shear yielding of angles	100 kip (448 kN) 105 kip (471 kN)
Shear rupture of angles Shear yielding of beam web	**91 kip (407 kN)** 96 kip (430 kN)

in combination in the real world. In an extreme scenario where the potential for horizontal beam web tear out could occur it would be limited to only the upper most bolt in the connection. It is important to note that a horizontal rupture of the beam web at the upper most bolt would not render this bolt completely ineffective in resisting a gravity load.

The difference between the two approaches would be still greater if the connection in question had been detailed with typical American bolt spacing of 75 mm (3 in.) rather than the 70 mm (2¾ in.) given in the Australian example problem. This greater bolt spacing increases the shear rupture capacity of the angles and actually causes beam web yielding to control the design (see Appendix B), at a strength of 430 kN (96 kips). Thus, the use of a 75 mm (3 in.) bolt spacing would result in an American design that has a capacity 78 kN (17 kips) greater than that of the Australian approach.

There are two other minor differences between the two design approaches that are noted when comparing the detailed calculation in Appendices B and C. These are the method of evaluating eccentric load effects on a bolt group, and the concern for shear flow in the clip angles. The Australian approach uses classic elastic theory for determining the effect of eccentric load on a bolt group, while American practice considers the instantaneous center of rotation. The concern for irregular geometry results in the Australian approach using a shear strength of 0.5 F_y for clip angles while the American approach uses 0.6 F_y. Neither one of these differences influences the final design of the connection.

CONCLUSION

The intent of this study is not to establish the appropriate limit states for a clip angle connection. There are numerous references that deal with this topic (Birkemoe and Gilmor, 1978; Munse, Bell and Chesson, 1959). The purpose is also not to demonstrate that one country's design approach is either superior to or less conservative than that of another country. What has been shown is that, given a structural design problem as common as a clip angle, two different approaches to engineering, based on training and experience, can interpret relatively similar design specifications, to produce significantly different results.

Table 4. Australian Limit States	
Limit State	**Strength Capacity**
Bolt shear	110 kips (494 kN)
Bolt bearing on angle	169 kips (758 kN)
Bolt bearing on web	107 kips (479 kN)
Shear rupture vertically on angle	138 kips (620 kN)
Shear rupture horizontally on web	**78 kips (352 kN)**
Shear yielding of angle	88 kips (393 kN)
Shear yield of beam web	91 kips (407 kN)

REFERENCES

Council of Standards (1998), *Australian Standard Steel Structures*, AS 4100, Council of Standards, Australia.

AISC (1989), *Manual of Steel Construction—Allowable Stress Design*, American Institute of Steel Construction.

AISC (2001), *Manual of Steel Construction Load and Resistance Factor Design*, 3rd Ed., American Institute of Steel Construction, November.

Hogan, T.J. and Thomas, I.R. (1994), *Design of Structural Connections*, 4th Ed., Australian Institute of Steel Construction.

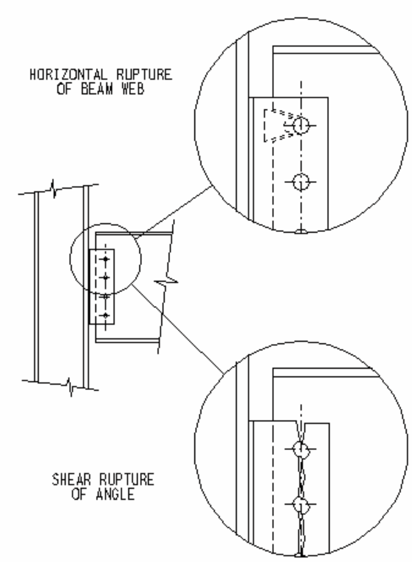

Fig. 2. Possible limit states.

Birkemoe, P.C. and Gilmor, M.L. (1978), "Behavior of Bearing Critical Double-Angle Beam Connection," *Engineering Journal*, American Institute of Steel Construction, Vol. 15, No. 4, 4th Quarter.

Munse, W.H., Bell, W.G., and Chesson, E. (1959), Behaviour of Riveted and Bolted Beam-to-Column Connections, *Journal of the Structural Division*, American Society of Civil Engineers, Vol. 85, No. ST3, March.

APPENDIX A

Comparison of Connection Designs AS4100 versus U.S. Practice (ASD/LRFD) (All references to ASD pertain to the 9th Ed. AISC ASD Manual (AISC, 1989) and all LRFD references pertain to the 3rd Ed. AISC LRFD Manual (AISC, 2001))

BOLTS

A comparison will be made for standard size high strength bolts. Typical structural bolts in Australia are M20 Grade 8.8. This fastener is comparable to a ¾-in. A325 bolt.

Material

AS/NZS 1252 Grade 8.8 Minimum tensile strength, f_{uf} = 830 MPa (120 ksi). Reference Table 9.3.1 of AS4100.
ASTM A325 Minimum tensile strength, F_u = 830 MPa (120 ksi) in diameters up to 1 in. Reference Table I-C, Section 4 of 9th Ed. ASD Manual.

Tension

AS 4100 Section 9.3.2.2

$N^*_{tf} \leq \phi N_{tf}$

where

$N_{tf} = A_s f_{uf}$

where

A_s = tensile stress area of the bolt = 0.334 in.^{2} per the Table on page 4-147 of ASD Manual

ϕ = 0.80 per Table 3.4 of AS 4100

N^*t_f = (0.334 in.^{2}) (120 ksi) 0.80 = 32 kips design capacity (ultimate)

LRFD Section J3.6, design tension capacity = $\phi F_t A_b$

where

A_b = nominal unthreaded area of bolt = 0.442 in.^{2} per the Table on Page 4-147 of ASD Manual

F_t = nominal tensile strength per LRFD Spec. Table J3.2 = 620 MPa (90 ksi)

ϕ = 0.75 per Table J3.2

(0.442 in.^{2}) (90 ksi) 0.75 = 30 kips design capacity (ultimate)

ASD Section J3.4 (Table J3.2) allowable load = $F_t A_b$

where

F_t = allowable tension 303 MPa (44 ksi)

A_b = nominal unthreaded area of bolt = 0.442 in.^{2} per the Table on page 4-147 of the ASD Manual

(0.442 in.^{2}) (44 ksi) = 19.4 kips design capacity (allowable)

Comparing AS 4100 with ASD: Load Factor = 32.0/19.4 = 1.65 > 1.5

Shear–Bearing

AS 4100, Section 9.3.2.1

$V^*_f \leq \phi V_f$

where

$V_f = 0.62 f_{uf} k_r (n_n A_c + n_x A_o)$

where

k_r = reduction factor for length of lap splice; reduction begins at 300 mm (12 in.), and has a maximum value of 0.75 at 1300 mm (50 in.); take as 1.0 for example

n_n = number of shear planes with threads included

n_x = number of shear planes with threads not included

A_c = minor diameter area = 0.309 in.^{2} per the Table on page 4-147 of ASD

A_o = nominal diameter area = 0.442 in.^{2} per the Table on page 4-147 of ASD

ϕ = 0.80 per Table 3.4 of AS 4100

SINGLE SHEAR THREADS INCLUDED
V^*_f = 0.80 (0.62) (120 ksi) (0.309 in.^{2}) = 18.4 kips design capacity (ultimate)

SINGLE SHEAR THREADS NOT INCLUDED
V^*_f = 0.80 (0.62) (120 ksi) (0.442 in.^{2}) = 26.3 kips design capacity (ultimate)

LRFD Section J3.6, design shear capacity = $\phi F_v A_b$. For splice connections greater than 1300 mm (50 in.), reduce capacity to 0.80.

where

A_b = nominal unthreaded area of bolt = 0.442 in.^{2} per the Table on page 4-147 of ASD

F_v = nominal shear strength per Table J3.2 = 330 MPa (48 ksi) for threads included in the shear plane and 414 MPa (60 ksi) for threads not included in the shear plane

ϕ = 0.75 per Table J3.2

SINGLE SHEAR THREADS INCLUDED
(0.442 in.^{2}) (48 ksi) 0.75 = 16 kips design capacity (ultimate)

SINGLE SHEAR THREADS NOT INCLUDED
(0.442 in.^{2}) (60 ksi) 0.75 = 20 kips design capacity (ultimate)

ASD Section J3.4 (Table J3.2) allowable load = $F_v A_b$ for splice connections greater than 1300 mm (50 in.) reduce capacity to 0.80.

where

F_v = allowable shear =145 MPa (21 ksi) for threads included in the shear plane and 207 MPa (30 ksi) for threads not included in the shear plane

A_b = nominal unthreaded area of bolt = 0.442 in.^{2} per the Table on Page 4-147 of ASD

SINGLE SHEAR THREADS INCLUDED
(0.442 in.^{2}) (21 ksi) = 9.3 kips design capacity (allowable)

Comparing AS 4100 with ASD: Load Factor = 18.4/9.3 = 1.98 > 1.5

SINGLE SHEAR THREADS NOT INCLUDED
(0.442 in.^{2}) (30 ksi) = 13.3 kips design capacity (allowable)

Comparing AS 4100 with ASD: Load Factor = 26.3/13.3 = 1.98 > 1.5

Shear–Slip Critical

AS 4100, Section 9.3.3.1

$V^*_{sf} \leq \phi V_{sf}$

where

$V_{sf} = \mu k_h n_{ei} N_{ti}$

where

K_h = factor for different holes = 1.0 for standard holes, 0.85 for oversize and short slots

n_{ei} = number of effective interfaces

N_{ti} = minimum bolt tension at installation = 131 kN (29 kips) estimated from Section 15.2.5.1 for a ¾ in. diameter (19 mm) bolt

μ = slip factor = 0.35 for clean as-rolled surface

ϕ = 0.70 per Section 3.5.5

SINGLE SHEAR STANDARD HOLE
V^*_{sf} = 0.70 (0.35) (1.0) (29 kips) = 7.1 kips design capacity (service load)

SINGLE SHEAR OVERSIZE OR SHORT-SLOTTED HOLE
V^*_{sf} = 0.70 (0.35) (0.85) (29 kips) = 6.0 kips design capacity (service load)

LRFD Section J3.8b (Appendix J) Design Shear Capacity = $\phi F_v A_b$

where

A_b = nominal unthreaded area of bolt = 0.442 in.^{2} per the Table on Page 4-147 of ASD

F_v = nominal slip critical shear resistance per Table J3.6 = 117 MPa (17 ksi) for standard hole and 103 MPa (15 ksi) for an oversize and a short-slotted hole based on a slip coefficient of 0.33 for clean mill scale

ϕ = 1.0 for standard oversize and short-slotted holes

SINGLE SHEAR STANDARD HOLE
(0.442 in.^{2}) (17 ksi) 1.0 = 7.5 kips design capacity (service load)

SINGLE SHEAR OVERSIZE OR SHORT SLOT HOLE
(0.442 in.^{2}) (15 ksi) 1.0 = 6.6 kips design capacity (service load)

ASD Section J3.4 (Table J3.2) Allowable Load = $F_v A_b$

where

F_v = allowable shear =117 MPa (17 ksi) for standard hole and 103 MPa (15 ksi) for oversize and short-slotted hole based on a slip coefficient of 0.33 for clean mill scale

A_b = nominal unthreaded area of bolt = 0.442 in.^{2} per the Table on Page 4-147 of ASD

SINGLE SHEAR STANDARD HOLE
(0.442 in.^{2}) (17 ksi) = 7.5 kips design capacity (allowable)

Comparing AS 4100 with ASD: Load Factor = 7.1/7.5 = 0.95 < 1.0 (this is a check at service load)

SINGLE SHEAR OVERSIZE OR SHORT-SLOTTED HOLE
(0.442 in.^{2}) (15 ksi) = 6.6 kips design capacity (allowable)

Comparing AS 4100 with ASD: Load Factor = 6.0/6.6 = 0.91 < 1.0 (this is a check at service load)

Shear Tension Interaction–Bearing

AS 4100 Section 9.3.2.3

$(V^*_f / \phi V_f)^2 + (N^{*tf} / \phi N_{tf})^2 \leq 1.0$

This is an elliptical interaction.

LRFD Section J3.7 (Table J3.5)—see commentary to specification. The equations in Table J3.5 provide a tri-linear approximation of an elliptical interaction.

ASD Section J3.5 (Table J3.3): This section requires an elliptical interaction.

Shear Tension Interaction–Slip Critical (service loads)

AS 4100 Section 9.3.3.3

$(V^*_{sf} / \phi V_{sf}) + (N^*_{tf} / \phi N_{tf}) \leq 1.0$

This is a linear interaction.

LRFD Section J3.9 requires reducing the shear resistance by the factor $(1 - T/0.8\, T_b N_b)$ where T is the service load tension, T_b is the minimum bolt pretension and N_b is the number of bolts. This is a linear interaction.

ASD Section J3.6 requires reducing the shear resistance by the factor $(1 - f_t A_b / T_b)$ where f_t is the average tensile stress applied to all of the bolts in the connection. This is a linear interaction.

Minimum Edge Distance

AS 4100 Section 9.6.2

1.75 diameters at sheared or hand flame-cut edges

1.5 diameters at machine flame-cut, sawn or planed edges

1.25 diameters at rolled edges

ASD & LRFD Table J3.5 (ASD) & Table J3.4 (LRFD)

Approximately 1.75 diameters at sheared edges (for ¾ in. diameter 1¼ in. edge distance)

Approximately 1.25 diameters at rolled gas cut or sawn edges (for ¾ in. diameter 1 in. edge distance)

Bearing at Bolt Holes

AS 4100 Section 9.3.2.4

$V^*_b \le \phi\, V_b$

$\phi = 0.9$ from Table 3.4

$V_b = 3.2\, d_f\, t_p\, f_{up}$ or $V_b = a_e\, t_p\, f_{up}$ whichever is less

where
- d_f = diameter of bolt
- t_p = thickness of plate
- a_e = edge distance from center of bolt hole
- f_{up} = ultimate strength of plate

LRFD Section J3.10

Strength = $\phi\, R_n$

where
$\phi = 0.75$

$R_n = 2.4\, d\, t\, F_u$ or $3.0\, d\, t\, F_u$ when deformation at the hole is not critical.

where
- d = diameter of bolt
- t = thickness of plate
- F_u = ultimate strength of plate

ASD Section J3.7 bearing and Section J3.9 edge distance

$F_p = 1.2\, F_u$ applied to the projected area of the bolt (i.e. bolt diameter times plate thickness)

$L_e \ge 2\, P/F_u\, t$ or $P = L_e\, t\, F_u / 2$

L_e = edge distance from center of hole

WELDS

Material

AS 4100 Table 9.7.3.10 (1)
E41XX nominal tensile strength 410 MPa (60 ksi)
E48XX nominal tensile strength 480 MPa (70 ksi)

AWS D1.1

E60XX nominal tensile strength 410 MPa (60 ksi)
E70XX nominal tensile strength 480 MPa (70 ksi)

Fillet Welds

AS 4100 Section 9.7.3.10
$V^*_w \le \phi\, V_w$ where $V_w = 0.6\, f_{uw}\, t_t\, k_r$

where
- ϕ = 0.80 for SP and 0.60 for GP per Table 3.4
- t_t = design throat thickness
- k_r = reduction factor to account for the length of a lap connection = 1.0 up to 1.7 m and reduces to a maximum of 0.62 for lengths up to 8.0 m
- f_{uw} = nominal tensile strength of weld metal

$\phi\, V_w = 0.8 \times 0.6 \times 480 \times 4.24 \times 1.0 = 0.98$ KN/mm

LRFD Table J2.5

Weld capacity = 0.75 (0.60) F_{EXX} applied to throat area or 0.90 F_y applied to base metal under leg.

F_{EXX} = minimum specified strength of weld metal

ASD Table J2.5

Weld capacity = 0.3 applied to throat area or "matching stress on base metal under leg."

Weld Capacity = 0.3 x 480 x 4.24 = 0.61 KN/mm

Comparing AS 4100 with ASD: Load Factor = 0.98/0.61 = 1.61 > 1.5

Partial Penetration Welds

AS 4100 Section 9.7.2.7
$V^*_w \le \phi\, V_w$

where
$V_w = 0.6\, f_{uw}\, t_t\, k_r$

where
- ϕ = 0.80 for SP and 0.60 for GP per Table 3.4
- t_t = design throat thickness
- k_r = reduction factor to account for the length of a lap connection = 1.0 up to 1.7 m
- f_{uw} = nominal tensile strength of weld metal

LRFD Table J2.5

Weld capacity = 0.75 (0.60) F_{EXX} for shear parallel to weld axis and 0.80 (0.60) F_{EXX} for tension normal to effective area or 0.90 F_y applied to base metal under weld.

where
F_{EXX} = minimum specified strength of weld metal

ASD Table J2.5

Weld capacity = 0.3 F_{EXX} applied to effective weld area or "matching stress on base metal under leg."

MINIMUM CONNECTION DESIGN LOAD

AS 4100 Section 9.1.4 (b)

For moment connection	=	0.50 times member moment capacity
For shear connection	=	40 kN (9 kip) or 0.15 times the member shear capacity
For axial connection	=	0.30 times the member tension/compression capacity

LRFD Section J1.7 44.5 kN (10 kip) minimum load

ASD Section J1.6 27 kN (6 kip) minimum load

Comparing AS 4100 with ASD:
Load Factor = 40.0/27.0 = 1.48 < 1.5

Comparing AS 4100 with ASD:
Load Factor = 0.98/0.61 = 1.61 > 1.5

APPENDIX B

AISC LRFD Check of Double Clip (Cleat) Angle (All references to LRFD pertain to the 3rd Ed. AISC Manual (AISC, 2001))

Eccentricity Effects

Based on instantaneous center of rotation.

C = Coefficient for eccentricity from LRFD Table 7-17
S = 70 mm say 3 in.
n = 4
e_x = 65 mm say 3 in.
C = 2.81 This is the effective number of bolts based on the given eccentricity.

Bolt Shear

From LRFD Table 7-10, double shear for a single ¾-in. bolt = 31.8 kips
Area of ¾-in. bolt = 0.442 in.^{2}
Area of M20 bolt = 0.487 in.^{2}
Therefore, double shear for M20 = 31.8 kips (0.487 /0.442) = 35 kips

Therefore the bolt design shear strength is $C \times 156 = 2.81 \times 156 = 438$ kN

Bolt Bearing

From Equation J3-2a, for an M20 in a standard hole the design bearing strength is:

$\phi\, 2.4\, d\, t\, F_u$ = 0.75 (2.4) (20 mm) (1 mm) (400 MPa) = 14.4 kN/mm

14.4 kN/mm of web or angle thickness × (410 MPa = F_u Aus / 400 MPa = F_u U.S.A.)

Angles:
6 mm × 2 angles × 14.4 kN/mm × (410/400) × 4 bolts = 708 kN

Web:
7.6 mm × 14.4 kN/mm × (410/400) × 4 bolts = 448 kN

Shear Yield of Angle

Section F2: $\phi\, V_n$ $\phi = 0.90$

$V_n = 0.6\, F_y\, A$ = 0.6 (260 MPa) (2 × 6 mm × 280 mm) / 1000 = 524 kN

$\phi\, V_n = 0.90 \times 524 = 471$ kN

Shear Yield of Web

$V_n = 0.6\, F_y\, A$ = 0.6 (260 MPa) (7.6mm x 403 mm) / 1000 = 477 kN

$\phi\, V_n = 0.9 \times 477 = 430$ kN

Shear Rupture of Angles

Section J4: $\phi\, R_n$ $\phi = 0.75$

Effective hole diameter = 22 mm + 2 mm = 24 mm

A_{net} = (280 mm – 4 bolts × 24 mm hole) × 2 angles × 6 mm thick = 2208 mm^2

$R_n = 0.6\, F_u\, A_{net}$ = 0.6 (410 MPa) (2208 mm^2) / 1000 = 543 kN

$\phi\, R_n = 0.75 \times 543 = 407$ kN Controls

Note on U.S. Detailing Practice

The typical bolt spacing used in the U.S. is 3 in. or 75 mm. This would change the net area of the angle and result in an increased shear rupture capacity. The length of the angle would change from 280 mm (3 × 70 mm bolt spacing + 2 × 35 mm edge distance) to 295 mm (3 × 75 mm bolt spacing + 2 × 35 mm edge distance)

A_{net} = (295 mm – 4 bolts × 24 mm hole) × 2 angles × 6 mm thick = 2388 mm^2

$R_n = 0.6\, F_u\, A_{net}$ = 0.6 (410 MPa) (2388 mm^2) / 1000 = 587 kN

$\phi\, R_n = 0.75 \times 587 = 440$ kN

Note that now the shear yield of the web will control the design at 430 kN.

APPENDIX C

AS 4100 Check of Double Cleat (Clip) Angle

Eccentricity Effects

Eccentricity effects are calculated for vertical (Z_b) and horizontal (Z_e) components.

Spacing = 70 mm   n = 4

Distance from bolts on beam to face of column, e_x = 65 mm

$Z_b = n / (1 + (6\, e_x / (n + 1)\text{ Spacing })^2\,)^{½} = 2.67$

$Z_e = (n + 1)\text{ Spacing} / 6\, e_x = 0.897$

Bolt Shear

A single M20 N in double shear threads included in the shear plane:

Shear Capacity = 2 ϕ 0.62 $F_u\, A_c$

Note: A_c = minor axis area for threads in shear plane

Shear Capacity = 2 (0.8) (0.62) (830 MPa) (225 mm^2) = 185.2 kN per bolt

Bolt Bearing

Bearing Capacity = ϕ 3.2 $d\, t\, F_u$ = 0.9 (3.2) (20 mm) (1 mm) (410 MPa) = 23.6 kN/mm

Angles:
6 mm × 2 angles × 23.6 kN/mm = 283 kN per bolt

Web:
7.6 mm × 23.6 kN/mm = 179.36 kN per bolt

Vertical Bolt Capacity

Bolts are controlled by, bearing on the beam web, and the eccentricity factor Z_b.

Bolt capacity = 179.36 kN (2.67) = 479 kN

Shear Yield of Angle

Shear Capacity = ϕ 0.5 $F_y\, A$
Note: reduced shear allowable due to “non-uniform” cross section

Shear Capacity = 0.9 (0.5) (260 MPa) (2 × 6 mm × 280 mm) = 393 kN

Shear Yield of Web

Shear Capacity = ϕ 0.6 $F_y\, A$

$A = t_w\, (d - 2\, t_f)$ = 7.6 mm (403 mm – 2 x 10.9 mm) = 2,897 mm^2

Shear Capacity = 0.90 (0.6) (260 MPa) (2897 mm^2) = 407 kN

Shear Rupture

Rupture Capacity = $\phi\, D_{critical}\, F_u$

$D_{critical}$ is taken as either the closest edge distance or the minimum spacing between bolt holes, whichever is less.

Angles Vertically:

$D_{critical}$ = 35 mm edge distance

Rupture Capacity = 0.9 (35 mm) (6 mm) (410 MPa) (2 angles) = 155 kN/bolt

Angles Horizontally:

$D_{critical}$ = 35 mm edge distance

Rupture Capacity = 0.90 (35 mm) (6 mm) (410 MPa) (2 angles) = 155 kN/bolt

Beam Web Vertically:

$D_{critical}$ = 70 mm bolt spacing – 22 mm hole diameter / 2 = 59 mm

Rupture Capacity = 0.90 (59 mm) (7.6 mm) (410 MPa) = 165 kN/bolt

Beam Web Horizontally:

$D_{critical}$ = 35 mm edge distance

Rupture Capacity = 0.9 (35 mm) (7.6 mm) (410 MPa) = 98 kN/bolt

Shear Rupture Vertically: (controlled by angles)

Capacity = 4 bolts (155 kN/bolt) = 620 kN

Shear Rupture Horizontally: (controlled by beam web)

Capacity = 4 bolts (Z_e) (98 kN/bolt) = 352 kN