1655

A Design Guide

for

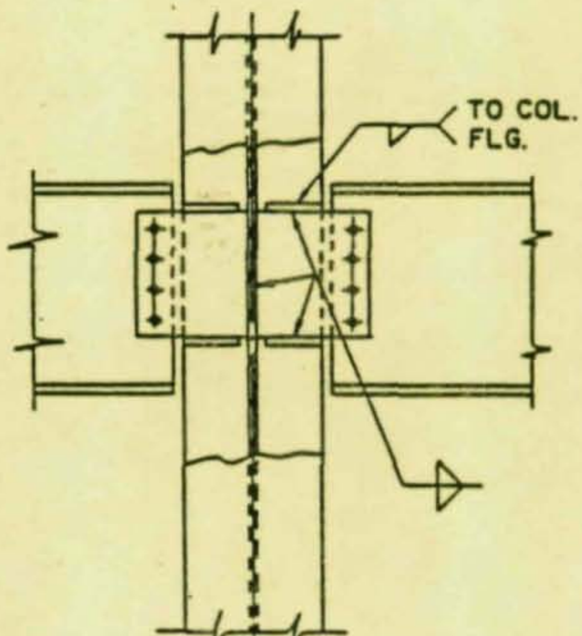
SINGLE PLATE FRAMING CONNECTION DESIGNS

Prepared for

THE AMERICAN INSTITUTE
of
STEEL CONSTRUCTION

by

Professor Ralph M. Richard, Ph.D., P.E.
Department of Civil Engineering
and Engineering Mechanics
THE UNIVERSITY OF ARIZONA
Tucson, Arizona 85721



July 1989

SINGLE PLATE FRAMING CONNECTION DESIGNS

A Design Guide

for

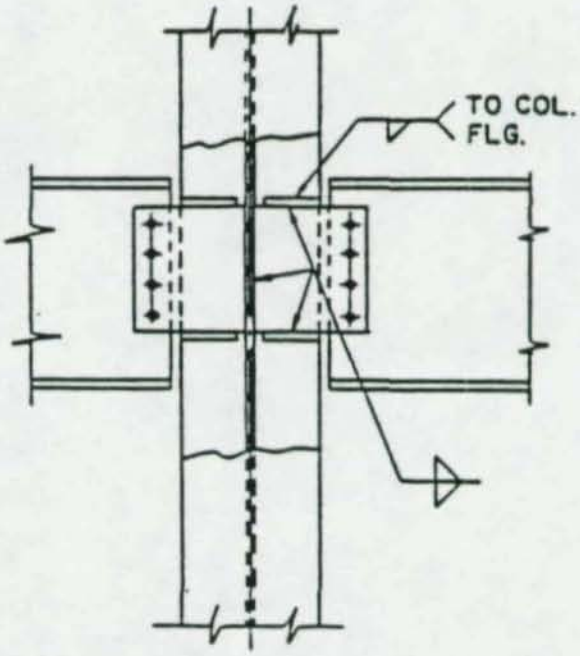
SINGLE PLATE FRAMING CONNECTION DESIGNS

Prepared for

THE AMERICAN INSTITUTE
of
STEEL CONSTRUCTION

by

Professor Ralph M. Richard, Ph.D., P.E.
Department of Civil Engineering
and Engineering Mechanics
THE UNIVERSITY OF ARIZONA
Tucson, Arizona 85721



July 1989

SINGLE PLATE FRAMING CONNECTION DESIGNS

PREFACE

An extensive theoretical and experimental investigation of single plate framing connections was performed during the period 1978-1982 in the research facilities of The University of Arizona, Tucson, Arizona. The principal investigator was Professor Ralph M. Richard. Professor James D. Kriegh was co-principal investigator, and a number of graduate students contributed significantly to the research effort.

This research was funded by the American Iron and Steel Institute and the American Institute of Steel Construction. Messrs. Ernest Hunter and Heinz Pak chaired the research committee that monitored the research.

Results of these investigations were published in the following *AISC Engineering Journals*.

"The Analysis and Design of Single Plate Framing Connections,"
by Ralph M. Richard, Paul E. Gillett, James D. Kriegh
and Brett A. Lewis
Vol. 17, No. 2 (1980).

"Design of Single Plate Framing Connections with A307 Bolts,"
by Ralph M. Richard, James D. Kriegh and David E. Hormby
Vol. 19, No. 4 (1982).
Discussion by Edward P. Becker and Ralph M. Richard
Vol. 22, No. 1 (1985).

"Single Plate Framing Connections with Grade-50 Steel and
Composite Construction,"
by David E. Hormby, Ralph M. Richard and James D. Kriegh
Vol. 21, No. 3 (1984).

"Design Aids for Single Plate Framing Connections,"
by Ned W. Young and Robert O. Disque
Vol. 18, No. 4 (1981).

In 1988 an extensive study was completed which lead to a simplified design procedure for single plates. The basis of this procedure is given in the design report "Simplified Single Plate Connection Designs" by Maker El Salti and Ralph M. Richard which was submitted to AISC in November of 1988. Chapter 1 of this design guide contains a brief history of the single plate connection and

Chapter 2 gives the design concepts and criteria for this connection. Given in Chapter 3 are Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) examples that cover most typical design office applications.

Appendix A gives the basis for the simplified design procedure and also repeats all of the design examples of Chapter 3 using this procedure which gives essentially identical single plate designs as the detailed published procedure. The structural engineer using this manual will generally use the simplified procedure and will refer to the general procedure only in the more unusual design cases. Appendices B and C are the ASD and LRFD Design Manual Weld Group Tables, respectively, which may be used to design typical single plate connection designs.

CONTENTS

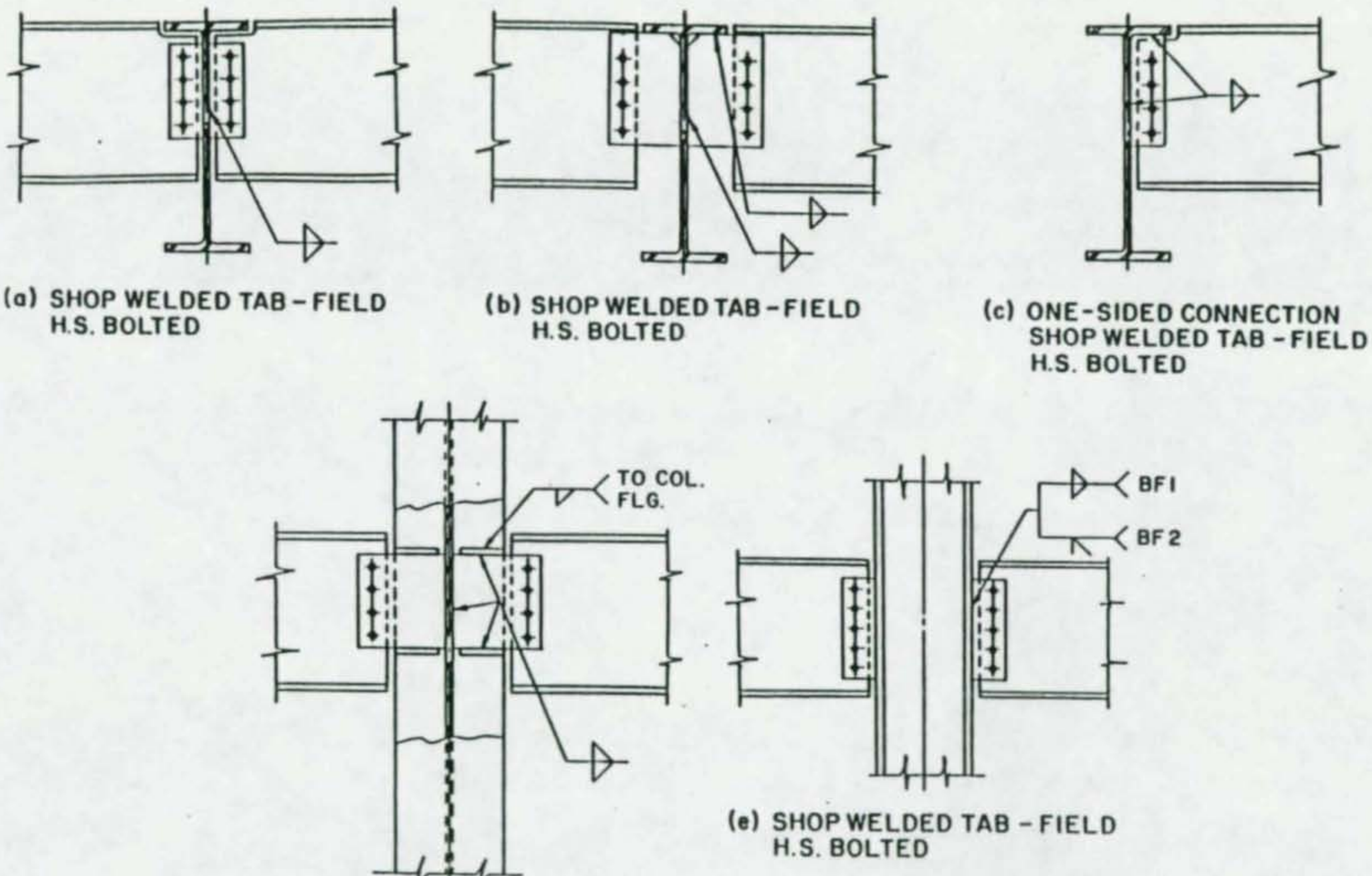
CHAPTER 1

HISTORY OF THE SINGLE PLATE CONNECTION

Single plate framing connections traditionally have been considered by structural steel designers to be a flexible "shear" connection. It is a very popular girder-to-column and beam-to-girder connection because it is economical to fabricate and results in simple field erection procedures. Typical single plate connections are shown in Figure 1. In all cases shown, the connection comprises a single plate with prepunched holes that is shop-welded to the supporting member. During erection the beam or girder with prepunched holes is brought into position and field-bolted to the framing plate. Unlike double framing angles which may have bolts in common with the angles for the beams in adjoining bays so that either erection bolts or erection angles may be required, all the single plate connection elements are independent of the others.

Prior to the research reported in References 1-3, the standard design procedure for this connection was to assume each bolt to share an equal portion of the total shear load, and in agreement with the simple support assumption, that relatively free rotation occurs between the end of the beam and the supporting member. Both the plate and weld were generally designed to resist the shear and, additionally, a moment equal to the shear times the distance from the bolt line to the weld. In fact, because of this simplified design procedure, the single plate connection was often called a "shear tab," "shear bar," or a "flag" connection. However, many structural engineers in the design profession recognized that this connection, unlike double framing angle connections which have elements subjected to flexure, generally lacked the ductility to accommodate rotations equal to that at the end of the simply supported beam as required by AISC Specifications.

An extensive research program at The University of Arizona established that sources of ductility were from (1) bolt deformation in shear, (2) plate and/or beam web hole distortion due to bolt bearing stresses, and (3) out-of-plate bending of the plate and/or beam web. Additional



(a) SHOP WELDED TAB - FIELD H.S. BOLTED

(b) SHOP WELDED TAB - FIELD H.S. BOLTED

(c) ONE-SIDED CONNECTION SHOP WELDED TAB - FIELD H.S. BOLTED

(d) SHOP WELDED TAB AND PLATES FIELD H.S. BOLTED

(e) SHOP WELDED TAB - FIELD H.S. BOLTED

Figure 1. Single Plate Framing Connections

connection rotation may occur from bolt slippage if the bolts are not in bearing at the time of initial loading. Because of these generally limited sources of ductility and the inherent rotational stiffness of this connection, it was also recognized that the moment at the weldment could be significantly larger than the simplified design procedure predicted.

Even though this connection has an apparent failure-free performance record, this does not indicate that previous design procedures were adequate. The actual force, stress, and strain distributions in the connection elements should be understood and the design methods and specifications should reflect the actual structural behavior so the factors of safety in Allowable Stress Design (ASD) and the resistance and load factors in Load and Resistance Factor Design (LRFD) are properly applied.

Lipson (5, 6) at the University of British Columbia reported results of full-scale tests which indicated that single plates can develop a significant connection moment. Caccavale (7), using the results of single shear bolt tests in plates of the same thickness used by Lipson, was able to analytically model the Lipson beam tests using an inelastic finite element program (8).

The research efforts at The University of Arizona sponsored by the American Iron and Steel Institute and the American Institute of Steel Construction from 1978 to 1982 that involved full-scale testing of beams and extensive inelastic finite element analyses have resulted in design procedures for a wide variety of single plate connection designs. These design procedures are applicable to the "shear" connection and do not apply when single plates are used in "moment" connections wherein single plates may be designed on the basis of the connection shear only.

CHAPTER 2

DESIGN CONCEPTS AND CRITERIA

2.1 INTRODUCTION

Presented in References 1, 2, and 3 is the research leading to recommended guides for the design of single plate framing connections. These procedures are applicable to the Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) design codes for connections using the following components and structural systems:

1) Noncomposite beams and composite beams, unshored and shored;

2) ASTM A36 and Grade 50 steel beams;

3) High-strength A325 and A490 bolts, snug tight or fully tightened, in standard round or slotted holes;

4) A307 bolts in standard round or slotted holes; and/or

5) ASTM A36 single plates.

In the American Institute of Steel Construction ASD and LRFD Specifications, it is stated that flexible (simple) beam connections shall be designed to accommodate the simply supported beam end rotation (ASD AISC 1.15.4 and LRFD J1.2). To accomplish this, inelastic action in the connection is permitted. The reason for this is illustrated in Figure 2 where typical connection moment rotation curves for ASD Type 1 (rigid), Type 2 (flexible), and Type 3 (semi-rigid) are shown along with a beam line to demonstrate typical connection moments and rotations. There are three popular Type 2 connections; these are: (1) the single plate, (2) double framing angles, and (3) the seated connection. Certain designs of all three of these connections can generate moments between 10 to 20 percent of the beam fixed-end moment for typical designs. This is shown in the example design problems presented herein for single plate connections and is shown in Reference 6 for double framing angle connections. The double framing angle connection and the seated connection, with a top angle required to provide lateral support of the compression flange, derive

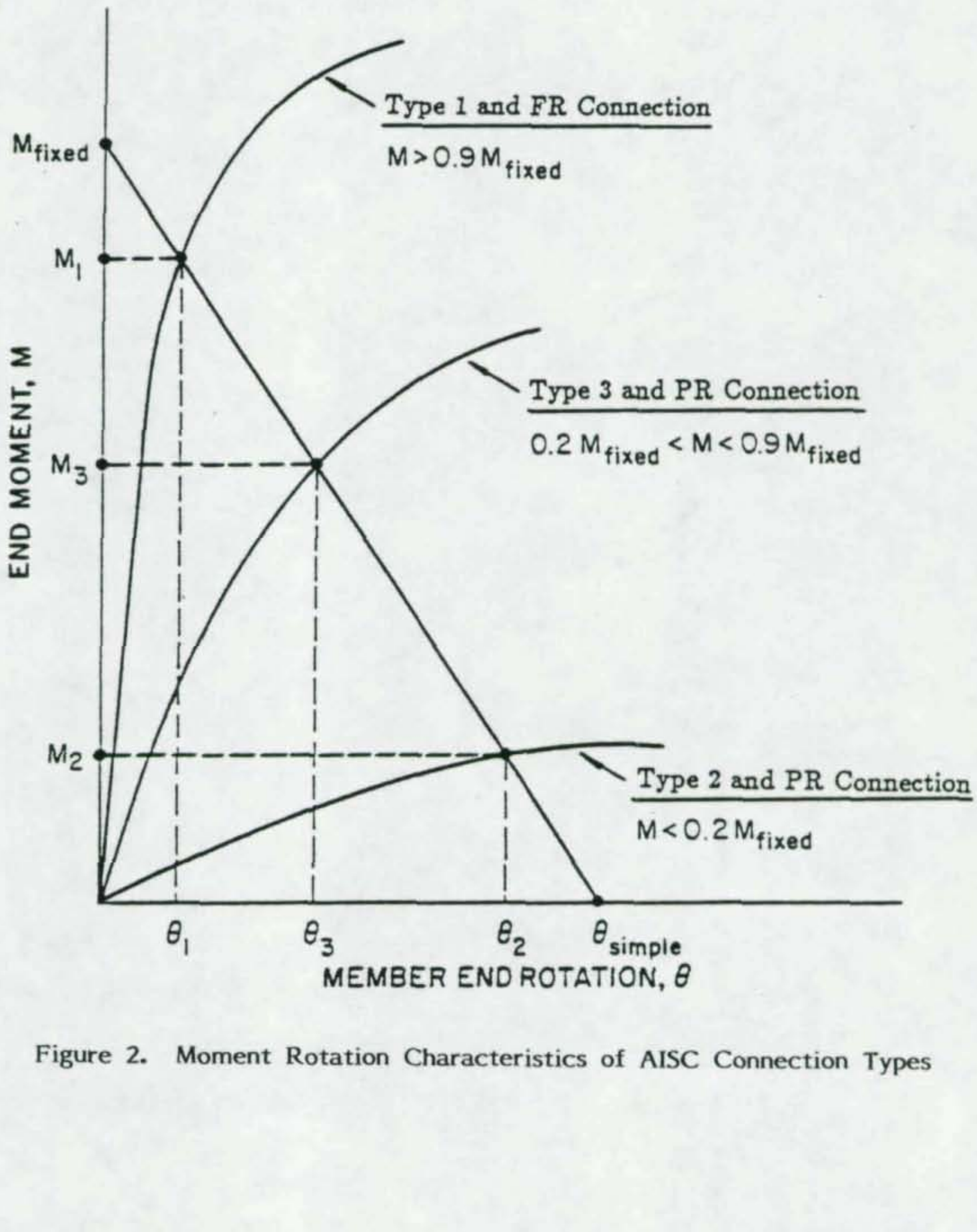


Figure 2. Moment Rotation Characteristics of AISC Connection Types

their ductility from angle elements in flexure. Shown in Figure 3 is a typical single plate connection design. The single plate, when designed using high-strength bolts in standard round holes derives its potential ductility, as shown in Figure 4, primarily from plate and/or beam web distortion due to the bolt bearing stresses. Alternatively, snug tight high-strength bolts in slotted holes or A307 bolts may be used to accommodate the beam rotation. If A307 bolts are used in standard holes, the maximum bolt distortion as shown in Figure 4 should be limited to approximately 0.10 inches. The A307 bolt, unlike A325 and A490 bolts, is very ductile and may often provide all of the necessary connection ductility required (7).

It has been common professional practice to neglect the effects of flexible connection moments and also the accompanying beneficial increased stiffening of the supporting structural component. However, it is important that the designer be aware of the effects of these moments and stiffnesses, which tend to beneficially offset each other, and be certain that all the elements of the connection have sufficient strength and ductility to accommodate the connection forces. For single plates, the most critical component of the connection is the weldment of the plate to the supporting structure as shown in Figure 3 where the connection moment distribution is given.

2.2 SINGLE PLATE CONNECTION DESIGN CRITERIA

There are three structural components to be designed in the single plate connection; these are: (1) the plate, (2) the bolts, and (3) the weld. Design criteria for each of these elements are given below.

2.2.1 Design Criteria for the Plate

2.2.1.1 Use ASTM A36 steel plate for ductility with a single row of bolts.

2.2.1.2 For either snug tight or fully tightened high-strength bolts in standard round holes, design ductility into the connection by providing the following geometric properties (refer to the research data presented in Reference 1):

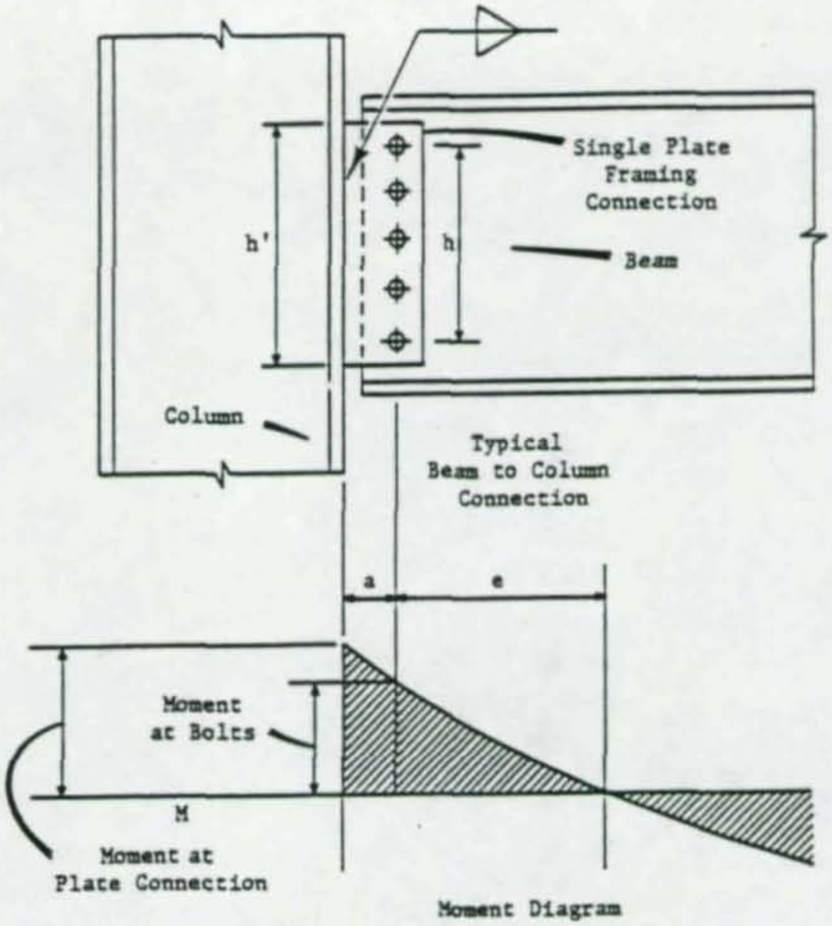


Figure 3. Single Plate Connection

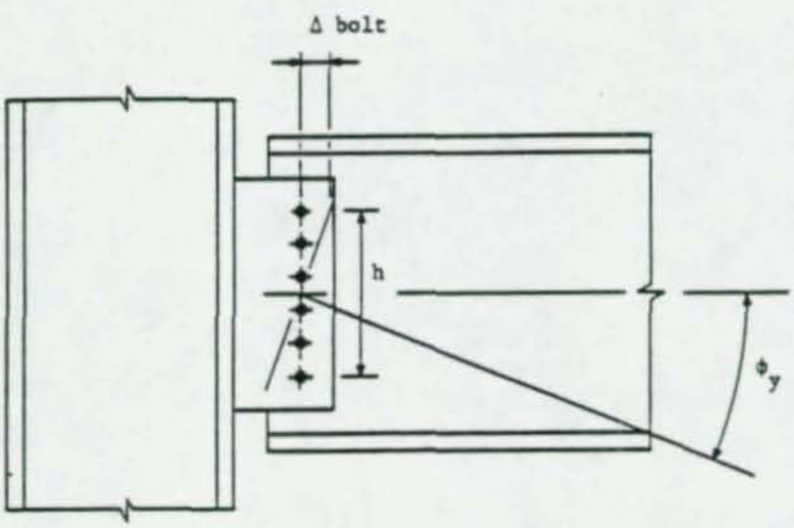
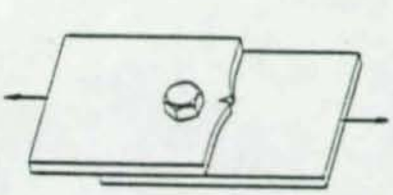


Figure 4. Connection Rotation With Bolt Deformation

$\frac{L_H}{D} > 2$ ... Required to prevent tension tearing.

$\frac{D}{t} > 2$ ... Required to prevent the bolt shear mode of failure for A325 bolts.



$\frac{D}{t} > 1.5$ ... Required to prevent the bolt shear mode of failure for A490 bolts.

where L_H = distance from the bolt center line to the edge of the plate and/or beam web,

D = bolt diameter, and

t = plate or beam web thickness, whichever is smaller.

If the beam is Grade 50, $t_{web\ equiv} = t_{web} \times \frac{50}{36}$

2.2.1.3 If the holes are slotted, constraints in 2.2.1.2 are not required, so use standard edge distances (AISC ASD Specification Table 1.16.5.1 and LRFD Table J3.7).

2.2.1.4 If the beam web thickness controls in 2.2.1.2 above, and the beam is coped, the block shear mode of failure should be checked. Generally, this failure mode will not control the design because the bolts are in single shear, but may if the connection depth is less than one-half the beam depth.

2.2.1.5 If A307 bolts are used in standard round holes, limit the maximum bolt distortion to less than 0.10 inches as shown in Figure 4. That is, maintain $\Delta_{top\ bolt} = \phi_{simple\ beam} \times h/2 < 0.10''$, where $\phi_{simple\ beam} = \frac{w\ell^3}{24EI}$ for uniformly distributed loads.

2.2.1.6 If snug tight A325 or A490 bolts are used in either short or long slotted holes, the center hole of the bolt pattern need not be slotted. This detail can be useful for alignment of the structure during erection.

2.2.2 Design Criteria for the Bolts

2.2.2.1 Compute the number of bolts required by dividing the beam shear by the allowable bolt load. This assumes equal shear in each bolt which is not true because of the moment at the bolt line as shown in Figure 3. However, if ductility is designed into the connection by preventing bolt shear and tension tearing of the plate and beam web, adequate connection strength exists at the bolt line.

2.2.2.2 Use only a single row of bolts.

2.2.3 Design Criteria for the Weldment

2.2.3.1 Compute the connection moment at the weld line as shown in Figure 3. This moment depends upon the number, size and specification of the bolts and the properties of the beam and loads. If high-strength bolts are used in slotted holes, the moment is dependent on whether the bolts are snug or fully tightened.

2.2.3.1.1 If fully tightened A325 or A490 bolts are used in either standard round or slotted holes, or if snug tight A325 or A490 bolts are used in standard holes, compute the beam eccentricity, e, as shown in Figure 3 as follows:

e = distance from bolt line to point of inflection of beam (eccentricity), inches.

$$= h \times \left(\frac{e}{h_{ref}}\right) \times \frac{n}{N} \times \left(\frac{S_{ref}}{S}\right)^{0.4}$$

h = distance between center of top and bottom bolts, inches

n = number of bolts

N = coefficient based on bolt diameter

= 5 for $\frac{3}{4}$ in. and $\frac{7}{8}$ in. bolts

= 7 for 1-in. bolts

S_{ref} = coefficient based on bolt diameter

= 100 for $\frac{3}{4}$ in. bolts

= 175 for $\frac{7}{8}$ in. bolts

= 450 for 1-in. bolts

S = section modulus of beam, inches^3

$\left(\frac{e}{h_{ref}}\right)$ = parameter based on $\frac{L}{d}$ ratio of beam

$$= 0.06 \frac{L}{d} - 0.15 \text{ when } \frac{L}{d} > 6$$

$$= 0.035 \frac{L}{d} \text{ when } \frac{L}{d} < 6$$

where L = span length of beam, in.

d = depth of beam, in. (total depth of composite beams.)

This formulation was developed for noncomposite and composite beams, either shored or unshored. For composite beams, substitute the transformed section modulus S_{tr} for S. There are three special considerations in the use of S_{tr}; these are: (1) when cover plates are used, (2) when the concrete stress governs the composite beam design, or (3) when partial composite designs are used. Refer to Reference 3, Page 133, when cover plates are used and Reference 8 for the latter case. Note that S may always be conservatively substituted for S_{tr}.

2.2.3.1.2 If A307 bolts in standard round holes or snug tight A325 or A490 bolts in slotted holes are used, compute the beam eccentricity as follows:

$$e = \left(\frac{n \times h}{384}\right)\left(\frac{L}{d}\right)$$

where n = number of bolts

h = depth of bolt pattern

L = length of beam

d = depth of beam (total depth of composite beams)

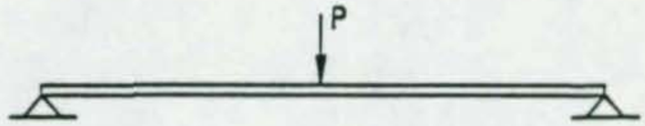
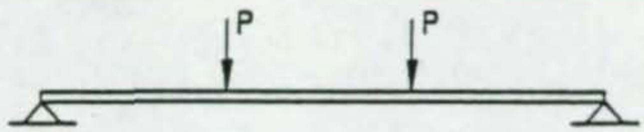
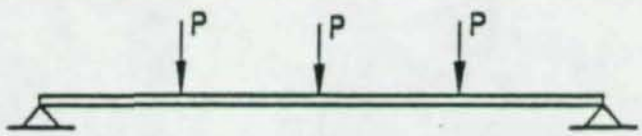
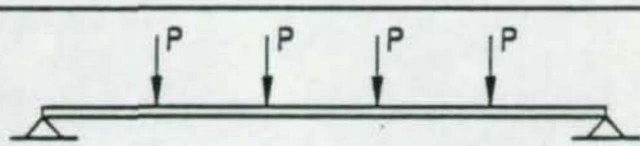
2.2.3.1.3 If the beam is of Grade 50 steel, reduce the eccentricity by the ratio of the steel strengths; that is,

$$e_{50} = e_{36} \times \frac{36}{50}$$

If the beam is not uniformly loaded, increase the eccentricity using the coefficients from Table 1 which is from Reference 1; that is,

$$e_{conc} = e_{uniform} \times \text{coefficient}$$

Table 1. Concentrated Load Eccentricity Coefficients

TYPE OF LOADING EQUAL LOADS, EQUAL SPACES	ECCENTRICITY COEFFICIENT
	2.00
	1.33
	1.33
	1.20

2.2.3.1.4 With the eccentricity known, the connection moment at the weld line is equal to

$$M = V \times (a + e)$$

where V = beam shear force

a = distance from the bolt line to the weld line as shown in Figure 3

e = eccentricity from 2.2.3.1.1 through 2.2.3.1.3

2.2.3.2 Check the plate bending and shear stresses:

For fully tightened high strength bolts

$$f_b = \frac{4M}{tb^2} < 22.\text{ ksi for fully tightened bolts (ASD)}$$

$$= \frac{4M}{tb^2} < 32.4\text{ ksi for fully tightened bolts (LRFD)}$$

or for A307 bolts or snug tight, high-strength bolts in slotted holes

$$f_b = \frac{6M}{tb^2} < 22.\text{ ksi (ASD)}$$

$$= \frac{6M}{tb^2} < 32.4\text{ ksi (LRFD)}$$

The shear stress is

$$f_v = \frac{V}{tb} < 14.4\text{ ksi (ASD)}$$

$$= \frac{V}{tb} < 20.2\text{ ksi (LRFD)}$$

where t = plate thickness

b = plate depth

2.2.3.3 Design the fillet welds based upon the resultant of the normal and shear stresses from 2.2.3.2. For example,

$$f_r = (f_b^2 + f_v^2)^{1/2}$$

$$\text{70XX weld req'd} = \frac{f_r x t}{2x0.93}\text{ sixteenths (ASD)}$$

$$= \frac{f_r x t}{2x1.39}\text{ sixteenths (LRFD)}$$

which gives the size of fillet welds on each side of the plate. Alternatively, the weld may be designed using the AISC Manual eccentrically loaded weld group design aids (see ASD Tables XIX, XXIII, and XXV, and LRFD Tables XVIII, XXII, and XXIV which are appended to this design guide).

2.3 BEAM L/d LIMITS AND END ROTATIONS

2.3.1 Noncomposite Beams

To ensure connection ductility by avoiding bolt shear and tension tearing of the plate or beam web when using high-strength bolts in standard holes, beam end rotations should be limited to a rotation that causes 0.2-in. deformation at the outermost bolts, as shown in Figure 4, at 1.5 times the working load for ASD or at the factored load for LRFD. To satisfy this requirement for noncomposite beams, the following limits on the L/d ratios are recommended:

$$\text{Noncomposite Beams } F_y = 36 \text{ ksi} \quad \frac{L}{d} < 36$$

$$\text{Noncomposite Beams } F_y = 50 \text{ ksi} \quad \frac{L}{d} < 24$$

Beams that exceed these limits can be evaluated by computing the simple beam end rotation and multiplying this by one-half the bolt pattern depth as shown in Figure 4.

2.3.2 Composite Beams

As with noncomposite beams with high-strength bolts in standard holes, it is recommended that beam end rotations be limited to the rotation that causes 0.2 in. of deformation at the outermost bolt as shown in Figure 4. The Commentary to the ASD AISC Specification Section 1.13.1 recommends a limit of $800/F_y$ on L/d for beams; this results in the following:

$$\frac{L}{d} < 22 \text{ for } F_y = 36 \text{ ksi}$$

$$\frac{L}{d} < 16 \text{ for } F_y = 30 \text{ ksi}$$

Although these limits are set to control deflections, they can be conservatively used to limit end rotations also.

2.3.3 Simple Beam End Rotations

The end rotation for simple beams is given by the formula

$$\phi = \frac{w\ell^3}{24EI}$$

where w = uniform load per unit length

ℓ = beam span

E = Young's Modulus

I = Beam Moment of Interia (Transformed for Composite Beams)

Typical end rotations are between 0.004 to 0.010 radians.

2.4 DESIGN AIDS

Reference 8, appended to this guide, contains extensive tables of single plate designs using fully tightened high-strength bolts for noncomposite beams. Table 2 from Reference 7 contains design data for single plates using A307 bolts in standard round holes or snug tight high-strength bolts in slotted holes for noncomposite and composite beams.

Weld designs may be made using the ASD and LRFD AISC Manual Tables on eccentrically loaded weld groups; specifically, the welds required for all of the single plate designs in Figure 1 may be sized using Tables XIX, XXIII, and XXV for ASD designs and Tables XVIII, XXII, and XXIV for LRFD designs.

Table 2. Limiting Beam Spans for A307 Bolts in Standard Holes (These span lengths are measured in feet.)

		NONCOMPOSITE BEAMS				COMPOSITE BEAMS			
		Allowable Bending Steel Stress				Allowable Bending Steel Stress			
Steel Beam	Number of A307 Bolts	22	24	30	33	22	24	30	33
W12	3	45.5	41.7	33.3	30.3	--	--	--	60.6
W16	4	40.4	37.0	29.6	26.9	--	--	59.2	53.8
W18	5	34.1	31.3	25.0	22.9	--	62.6	50.0	45.8
W21	6	31.8	29.2	23.3	21.2	63.6	58.4	46.6	42.4
W24	7	30.3	27.8	22.2	20.2	60.6	55.6	44.4	40.4

2.5 SUPPORT STRUCTURE DESIGN CONSIDERATIONS

A typical single plate-to-column connection design for mixed construction is shown in Figure 5 where headed studs, welded to a support plate, are used to resist the single plate shear and bending moment. In both ASD and LRFD this connection design should be based upon the ultimate strength of the headed studs by using the factored single plate moment and shear at the weld line. For ASD use 1.5 times the service load moment and shear.

When the single plate is welded to the wall of a tube column, as shown in Figure 6, the wall thickness should be checked to determine if it is thick enough to support the single plate moment and shear force. Presented in Reference 10 is a yield line analysis which may be conservatively used to assess the tube wall strength for this connection. Shown in Figure 7 is a yield line pattern used to check the tube wall strength. The notation used in the following derivation is also given in Figure 7.

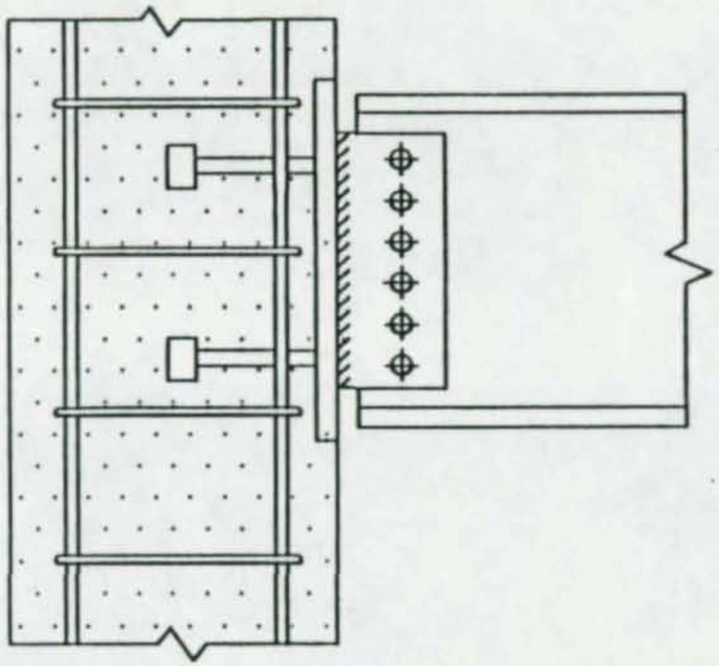


Figure 5. Single Plate Connection Design for Mixed Construction

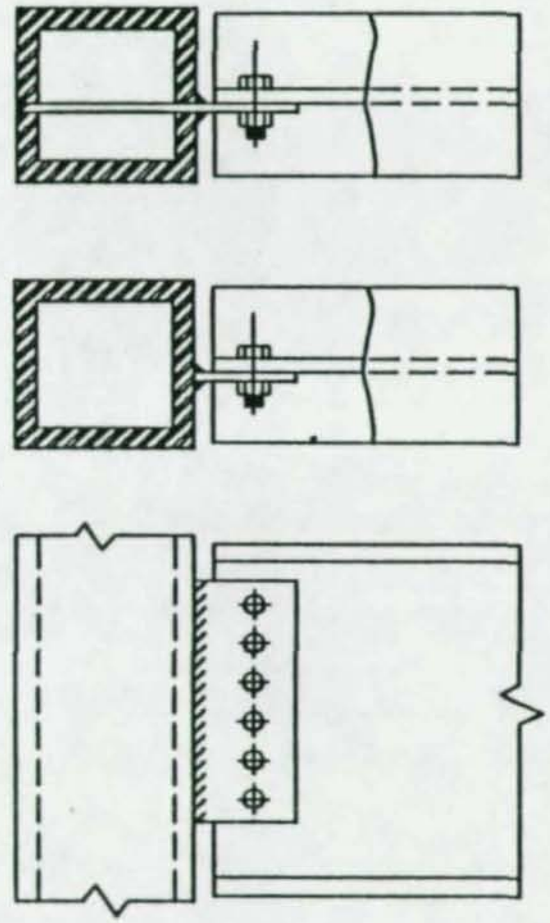
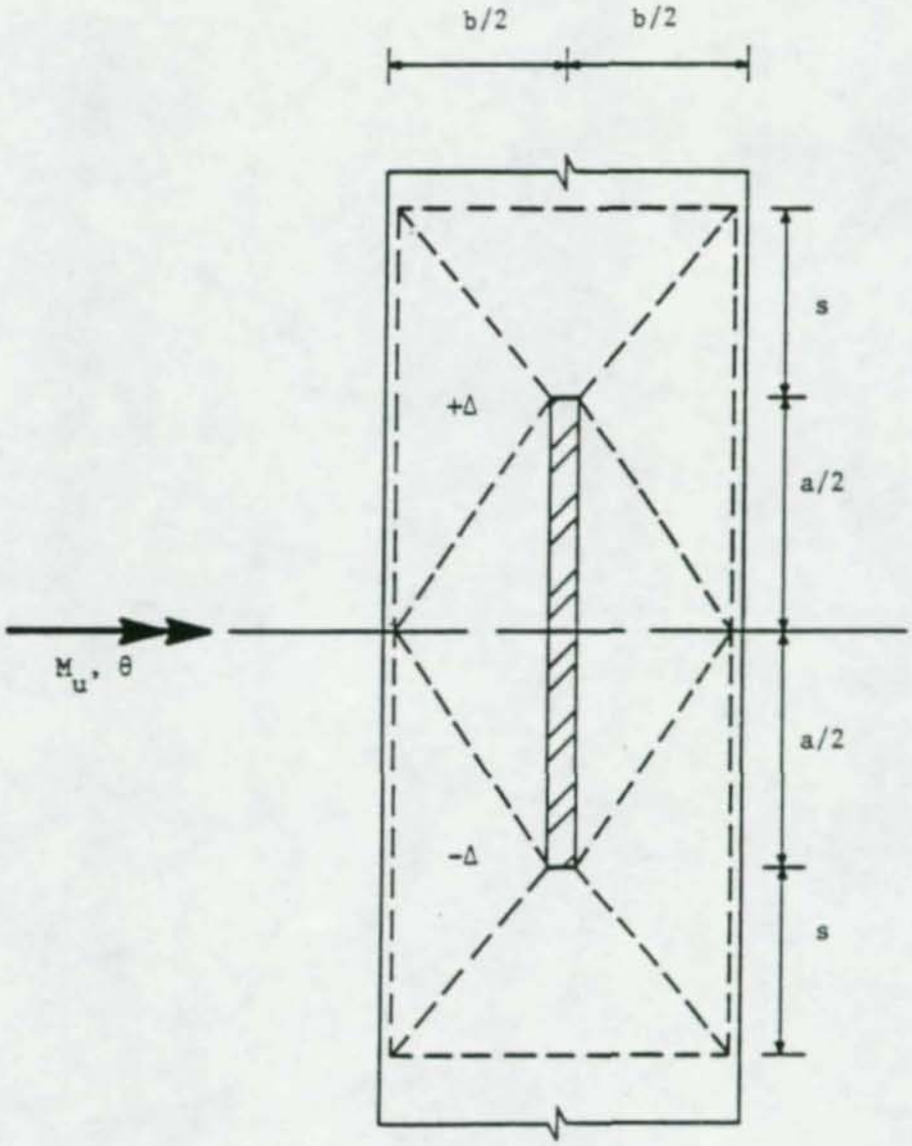


Figure 6. Single Plate To Tube Column Connection Designs



Notation

b = width of tube

a = depth of single plate

s = yield line length variable

M_u = factored single plate moment

θ = plate rotation

Δ = plate end deflection

Figure 7. Tube Column and Column Web Yield Line Model

Yield lines emanate from the ends of the plate to the intersection of the axes of roation which are along the edges as shown. Yield lines are also generated along these axes of rotation.

The virtual work theorem may be conveniently used here to determine the wall thickness. The external virtual work is:

$$E.\ W. = M_u\theta = \frac{2m_u\Delta}{a}$$

where θ is virtual rotation of the factored moment, M_u, about the centerline and Δ is the resulting virtual displacement at the ends of the plate. The internal virtual work is:

$$I.\ W. = 2m_u \left[4\left(s + \frac{a}{2}\right)\frac{2\Delta}{b} + \frac{2b\Delta}{s} + \frac{2b\Delta}{a}\right]$$

where m_u is the plastic bending moment per unit length of tube wall.

Equating the external virtual work to the internal virtual work gives the following equation:

$$M_u = m_u a\left[\frac{8s}{b} + \frac{4a}{b} + \frac{2b}{s} + \frac{2b}{a}\right]$$

To determine the variable s, minimize M_u with respect to s; i.e.,

$$\frac{dM_u}{ds} = m_u a\left[\frac{8}{b} + 0 - \frac{2b}{s^2} + 0\right] = 0$$

and solve for $s = \frac{b}{2}$

Since

$$m_u = \frac{F_y t^2}{4}$$

where F_y = yield strength of the steel

t = thickness of tube wall

then

$$t_{req'd} = \left[\frac{M_u}{F_y a\left(\frac{a}{b} + \frac{b}{2a} + 2\right)}\right]^{1/2}$$

EXAMPLE: A single plate is to be used to connect a W24x68 beam to a 10x10 in. tube column with a wall thickness of 5/8 in. The connection design results in a plate 18-in. deep with a service load moment at the weldment of 600 in.-k. Determine if the tube wall is thick enough to support this plate moment. The tube has a yield strength of 46 ksi, and the E70XX welds are 3/8-in. fillets.

Check Tube Wall Thickness

The factored moment for ASD is 1.5 times the service load moment,

$$M_u = 1.5 \times 600 = 900 \text{ in.-k}$$

so

$$t_{req'd} = \left[\frac{900}{46 \times 18\left(\frac{18}{10} + \frac{10}{36} + 2\right)} \right]^{1/2}$$

$$= 0.516 < 0.625 \text{ in.} \quad \text{O.K.}$$

Check the Effective Weld Size (See Reference 11, p. 214)

$$a_{max\ eff} = 1.89 \left[F_y \ \frac{t_{wall}}{F_u} \right]$$

$$= 1.89 \left[46 \times \frac{0.625}{70} \right]$$

$$= 0.78 > 0.375 \text{ in.} \quad \text{O.K.}$$

If $t_{req'd}$ were greater than t_{wall}, a WT could be substituted for the single plate. Alternatively, the single plate could be extended through the tube column as shown in Figure 6 where with the moment and shear at the bolt line known, the welds on the near and far side of the tube may be designed using ASD Table XIX or LRFD Table XVIII.

A first-order, conservative, adjustment in the plastic moment capacity in the tube wall due to an axial load in the column can be made by replacing F_y in the above formula with $\overline{F}_y$ where

$$\overline{F}_y = F_y \left[1 - \left(\frac{P}{P_y} \right)^2 \right]$$

where P = column axial load

P_y = column yield load

When a single plate is welded only to the girder web (as shown in Figure 1a) to form a one-sided connection that is not welded to the girder flange (as shown in Figure 1c) and the plate depth is significantly less than the girder depth, the girder web thickness should be checked to determine if it is thick enough to support the single plate bending moment and shear. An approximate yield line model with notation is shown in Figure 8. For this model

$$M_u = 4m_u \frac{d(2d - a)}{d-a}$$

Since

$$m_u = \frac{F_y t^2}{4}$$

Then

$$t_{req'd} = \left[\frac{M_u}{F_y \left(\frac{d(2d - a)}{d - a} \right)} \right]^{1/2}$$

EXAMPLE: A coped W24x68 beam with a single plate connection frames into a W30x108 girder (t_{web} = 0.545 in.) to form a one-sided connection similar to Figure 2c; however, the plate is not welded to the top flange of the girder. Check the web of the A36 steel girder for adequate strength. The factored moment is 900 in.-k and the end shear is 51 kips. The plate is 18-in. deep with 3/8-in. fillet welds. With a = 18 in., d = 28 in., and F_y = 36 ksi

$$t_{req'd} = \left[\frac{900}{36 \left(\frac{28(56-18)}{28 - 18} \right)} \right]^{1/2} = 0.485 \text{ in.}$$

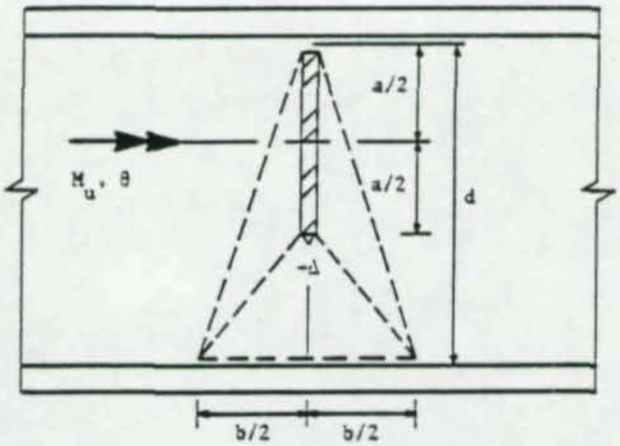
$$= 0.485 < 0.545 \text{ in. O.K.}$$

Check effective weld size as in the tube column example.

References 12 and 13 present similar yield line analyses.

Remarks

When one-sided single plate connections are designed, the flexibility of the supporting structure can significantly reduce the connection moment. It is beyond the scope of this paper to include the relative flexibility of the beam and connection to the supporting structure; however, using the basic equations of structural mechanics, this flexibility may often be easily estimated and used to develop economical connection designs.



Notation

d = depth of girder web M_u = factored single plate moment
a = depth of single plate θ = plate rotation
b = yield line variable length Δ = plate end displacement

Figure 8. Girder Web Yield Line Model

Alternatively, when one-sided single plate connections are used to connect beams to spandrel girders which, for example, support a panel or curtain wall, the connection stiffness and moment capacity may be sufficient to counter the moment in the outrigger beam supporting the wall. To evaluate the connection stiffness and strength, the moment-rotation data for the single plates may be determined from Reference 1.

In composite beam design, if negative reinforcement over the girder is used as shown in Figure 9, the single plate moment may be reduced significantly.

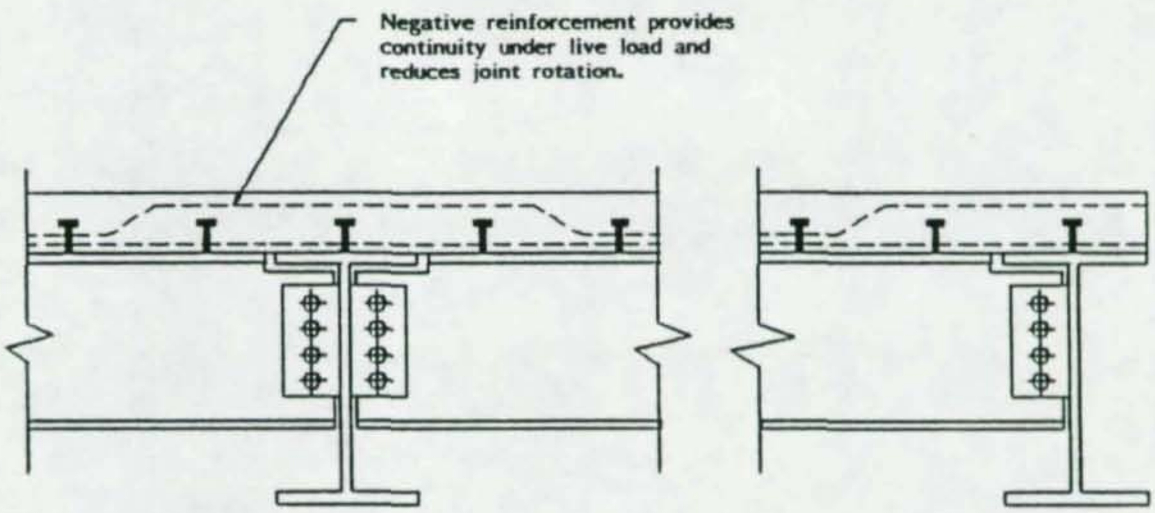
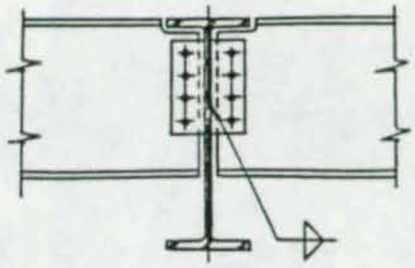


Figure 9. Composite Beam Connections

CHAPTER 3

SINGLE PLATE CONNECTION DESIGN EXAMPLES

ASD DESIGN EXAMPLE 1



Beam: W24 x 68, A36 Steel, $S = 154\ in^3$
Span: 24', Laterally Supported
Loading: Uniform Load with $W = 102^k$

ASD Design Procedure

Step

1 Select A36 plate with $t_{plate} = 3/8"$ ($t_{web} = 0.415"$)

2 Try 3/4" A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right) / \left(\frac{3}{8}\right) = 2.0$$

$$R = \frac{102}{2} - 51^k$$

$$n_{req'd} = \frac{51^k}{9.28^k} - 6 \text{ bolts}$$

3 $$\left(\frac{e}{h}\right)_{ref} = 0.06\ \frac{L}{d} - 0.15 = 0.57$$

$$\left(\frac{e}{h}\right) = 0.57 \times \frac{6}{5} \times \left(\frac{100}{153}\right)^{0.4} = 0.577$$

With pitch = 3", h = (6 - 1) x 3 = 15"
e = 0.577 x 15 = 8.65"

4 For a = 3", $V = R = 51^k$
M = 51 x (8.65 + 3) = 594.4 in.-k

5 $$f_b = \frac{4 \times 589}{0.375 \times 18^2} = 19.6 \text{ ksi} < 22 \text{ ksi}$$

$$f_v = \frac{51}{0.375 \times 18} = 7.56 \text{ ksi} < 14.4 \text{ ksi}$$

6 $$f_r - (19.4^2 + 7.56^2)^{1/2} = 20.98 \text{ ksi}$$

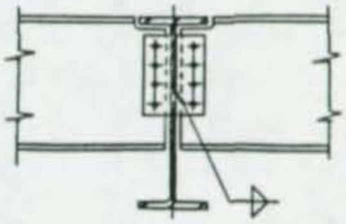
$$\text{70XX weld req'd} = \frac{20.8 \times 0.375}{0.93} = 8.5 \text{ sixteenths}$$

Use 5/16" fillets each side.

7 Alternate Weld Design. Use ASD AISC Table XIX.
$\ell = 18"$, $a\ell = 11.65$, $a = 11.65/18 = 0.647$, $C = 0.632$

$$\text{70XX weld req'd} = \frac{51}{1. \times 0.632 \times 18} = 4.48 \text{ sixteenths}$$

Use 5/16" fillets each side.



ASD DESIGN EXAMPLE 2

Beam: W24 x 62, A572 Grade 50 Steel, $S = 131 \text{ in}^3$
Span: 24', Laterally Supported
Loading: Uniform Load with $W = 120^k$

ASD Design Procedure

Step

1 Select A36 plate with $t_{plate} = 7/16"$ ($t_{web} = 0.430"$)
(t_{web} = equiv = 0.430 x 50/36 = 0.597")

2 Try 7/8" A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{7}{8}\right)/\left(\frac{7}{16}\right) = 2.0$$

$$R = \frac{120}{2} = 60^k$$

$$n_{req'd} = \frac{60^k}{12.63^k} = 5 \text{ bolts}$$

3 $$\left(\frac{e}{h}\right)_{ref} = 0.06\ \frac{L}{d} - 0.15 = 0.57 \text{ (From A36 Design Curve)}$$

$$\left(\frac{e}{h}\right) = 0.57 \times \frac{5}{5} \times \left(\frac{175}{131}\right)^{0.4} = 0.64$$

With pitch = 3", h = (5 - 1) x 3 = 12", and F_y = 50 ksi

$$e = 0.642 \times 12 \times \frac{36}{50} = 5.53$$

4 For a = 3", $V = R = 60^k$
M = 60 x (5.53 + 3) = 512 in.-k

5 $$f_b = \frac{4 \times 512}{0.4375 \times 15^2} = 20.8 \text{ ksi} < 22.0 \text{ ksi}$$

6 $$f_v = \frac{60}{0.4375 \times 15} = 9.14 \text{ ksi} < 14.4 \text{ ksi}$$

7 $$f_r = (20.8^2 + 9.14^2)^{1/2} = 22.6 \text{ ksi}$$

$$\text{70XX weld req'd} = \frac{22.6 \times 0.4375}{0.928} = 10.7 \text{ sixteenths}$$

Use 3/8" fillets each side.

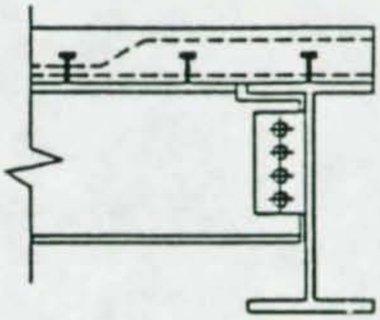
8 Alternate Weld Design. Used ASD AISC Table XIX.

$$\ell = 15,\ a\ell = 8.53,\ a = \frac{8.55}{15} = 0.57,\ C = 0.71$$

$$\text{70XX weld req'd} = \frac{60}{1. \times 0.71 \times 15} = 5.63 \text{ sixteenths}$$

Use 3/8" fillets each side.

ASD DESIGN EXAMPLE 3



Beam: W16 x 40, A36 Steel with 4" Slab, $S_{tr} = 92.9$ in³
Span: 24', Laterally Supported
Loading: Uniform Load with $W = 61.9^k$

ASD Design Procedure

Step

1 Select A36 plate with $t_{plate} = 5/16"$ ($t_{web} = 0.307"$)

2 Try 3/4" A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right) / \left(\frac{5}{16}\right) = 2.4 > 2.0$$

$$R = \frac{61.9}{2} = 30.9^k$$

$$n_{req'd} = \frac{30.9^k}{9.28^k} = 4 \text{ bolts}$$

3 $$\left(\frac{e}{h}\right)_{ref} = 0.06\ \frac{L}{d} - 0.15 = 0.714$$

$$\left(\frac{e}{h}\right) = 0.714 \times \frac{4}{5} \times \left(\frac{100}{92.9}\right)^{0.4} = 0.589$$

With pitch = 3", $h = (4 - 1) \times 3 = 9"$, and $F_y = 36$ ksi
$e = 0.589 \times 9 = 5.30"$

4 For $a = 3"$, $V = R = 30.9^k$
$M = 30.9 \times (5.30 + 3) = 256$ in.-k

5 $$f_b = \frac{4 \times 256}{0.3125 \times 12^2} = 22.8 \text{ ksi} < 24 \text{ ksi}$$

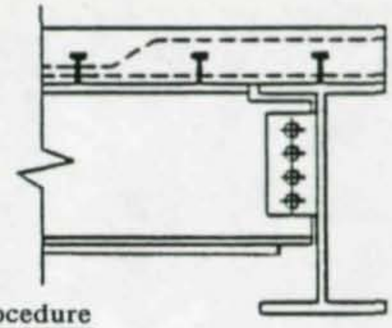
6 $$f_v = \frac{30.9}{0.3125 \times 12} = 8.24 \text{ ksi} < 14.4 \text{ ksi}$$

7 $f_r = (22.8^2 + 8.24^2)^{1/2} = 24.2$ ksi

$$\text{70XX weld req'd} = \frac{24.2 \times 0.3125}{0.93} = 8.13/\text{sixteenths}$$

Use 5/16" fillets each side.

ASD DESIGN EXAMPLE 4



Beam: W21 x 44 with 5-1/2" x 1/2" plate.
A572 Grade 50 Steel with 4" slab.
$S_t = 481$ in^3, $S_{tnp} = 406$ in^3
(Refer to Ref. 3, p. 133)

Span: 30', Laterally Supported

Loading: Uniform with $W = 130^k$

ASD Design Procedure

Step

1 Select A36 plate with $t_{plate} = 3/8"$ [$t_{web} = 0.348"$)
$t_{equiv} = 0.348 \times (50/36) = 0.483$, so A36 plate controls]

2 Try 3/4" A490N bolts either snug or fully tightened in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right) / \left(\frac{3}{8}\right) = 2.0$$

$$R = \frac{130}{2} = 65.0^k$$

$$n_{req'd} = \frac{65}{12.4} = 6 \text{ bolts}$$

3 $$\left(\frac{e}{h}\right)_{ref} = 0.06\ \frac{L}{d} - 0.15 - 0.697$$

$$\left(\frac{e}{h}\right) = 0.697 \times \left(\frac{6}{5}\right) \times \left(\frac{100}{481}\right)^{0.4} \times \left(\frac{406}{481}\right)^{1/2}$$

$= 0.410$

With pitch = 3", $h = (6-1) \times 3 = 15"$
$e = 0.410 \times 15 = 6.15$

4 For $a = 3"$, $V = R = 65.0^k$
$M = 65.0 \times (8.61 + 3) = 594$ in-k

5 $$f_b = \frac{4 \times 594}{0.375 \times 18^2} = 19.57 \text{ ksi} < 24 \text{ ksi}$$

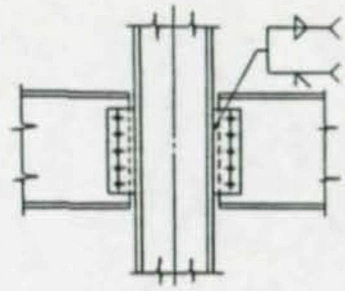
6 $$f_v = \frac{65.0}{0.375 \times 18} = 9.63 \text{ ksi} < 14.3 \text{ ksi}$$

7 $f_r = (19.57^2 + 9.63^2)^{1/2} = 21.8$ ksi

$$\text{700XX weld req'd} = \frac{21.8 \times 0.375}{0.93} = 8.79 \text{ sixteenths}$$

Use 5/16" fillets each side

ASD DESIGN EXAMPLE 5



Beam: W16 x 40, A36 Steel
Span: 24', Laterally Supported
Loading: Uniform Load with $W = 52^k$

ASD Design Procedure

Step

1 Select A36 plate with $t_{plate} = 5/16"$ ($t_{web} = 0.305"$)

2 Try 7/8" A307 bolts in standard holes

$R = \frac{52}{2} = 26^k$

$n_{req'd} = \frac{26}{6.0^k} = 5$ bolts

3 For 3" pitch, h = 12". At 1.5 times working load,

$\Delta_{top\ bolt} = 1.5 \times \frac{W\ell^2}{24EI} \times \frac{h}{2} = \frac{1.5 \times 52 \times (24 \times 12)^2}{24 \times 30 \times 10^3 \times 518} \times \frac{12}{2} = 0.103$

$\simeq 0.10"$, say O.K.

$e = \left(\frac{5 \times 12}{384}\right)\left(\frac{20 \times 12}{16}\right) = 2.34$

4 For a = 3", V = R = 26

$M = 26 \times (2.34 + 3) = 139$ in.-k

5 $f_b = \frac{6 \times 139}{0.316 \times 15^2} = 11.86 \text{ ksi} < 22.0 \text{ ksi}$

6 $f_v = \frac{26}{0.316 \times 15} = 5.55 \text{ ksi} < 14.4 \text{ ksi}$

7 $f_r = (11.86^2 + 5.55^2)^{1/2} = 13.09$ ksi

70XX weld req'd $= \frac{13.09 \times 0.316}{0.93} = 4.41$ sixteenths

Use 3/16" fillets each side.

ASD DESIGN EXAMPLE 6 (Refer to Figure 1c and Example 1)

Beam: W24 x 68, A36 Steel, $S = 154$ in³
Span: 24', Laterally Supported
Loading: Uniform Load with $W = 102^k$
Girder: W30 x 132

ASD Design Procedure

Step

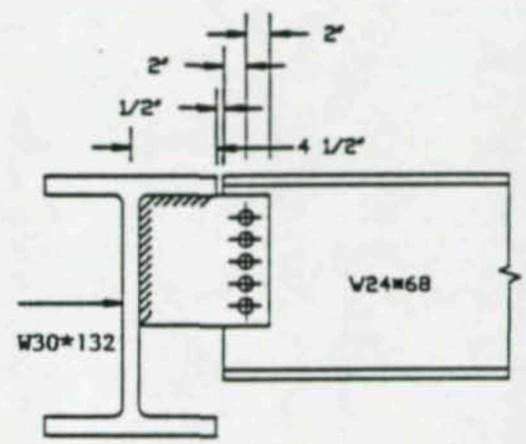
1 Select A36 plate with $t_{plate} = 3/8"$ ($t_{web} = 0.416"$)

2 Try 3/4" A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right) / \left(\frac{3}{8}\right) = 2.0$$

$$R = \frac{102}{2} = 51^k$$

$$n_{req'd} = \frac{51^k}{9.28^k} = 6 \text{ bolts}$$



3

$$\left(\frac{e}{h}\right)_{ref} = 0.06\ \frac{L}{d} - 0.15 = 0.57$$

$$\left(\frac{e}{h}\right) = 0.57 \times \frac{6}{5} \times \left(\frac{100}{153}\right)^{0.4} = 0.577$$

With pitch = 3", $h = (6 - 1) \times 3 = 15"$
$e = 0.577 \times 15 = 8.65"$

4 Allow 1/2" clearance between girder and beam flanges. With 2" edge distance in plate and beam web, and 1-1/2" end distances, use 8-3/4 x 3/8 x 1'-6" plate.

5 Check plate stresses at end of beam
$M = 51 \times (8.65 + 2.0) = 544$ in.-k

$$f_b = \frac{4 \times 544}{0.375 \times 18^2} = 17.9 \text{ ksi} < 22 \text{ ksi}$$

$$f_v = \frac{51}{0.375 \times 18} = 7.56 \text{ ksi} < 14.4 \text{ ksi}$$

6 From ASD AISC Table XXV with $\ell = 18"$ and $K\ell = 4\text{-}1/2"$, then $x = 0.025$
Therefore $x\ell = 0.45$
Now $a\ell = 8.65 + 2 + 1/2 + 6 - 0.45 = 16.75"$, so that $a = 0.93$
From Table XXV; $C = 0.310$

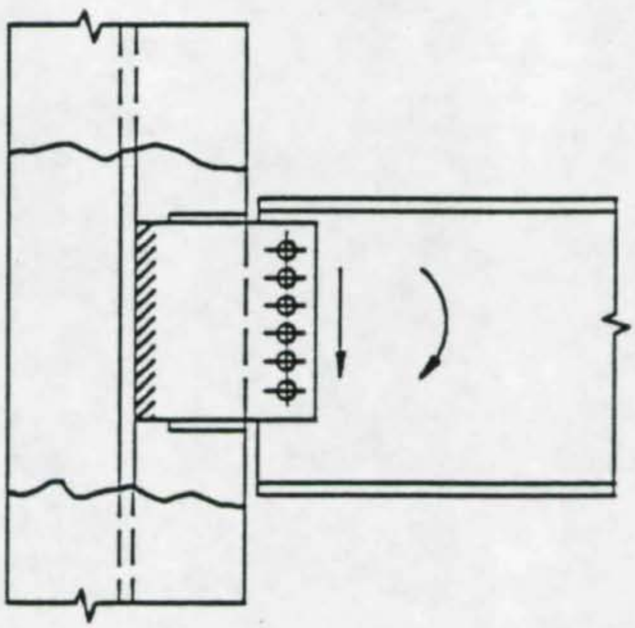
$$\text{70XX weld req'd} = \frac{51}{1 \times 0.310 \times 18} = 9.14 \text{ sixteenths}$$

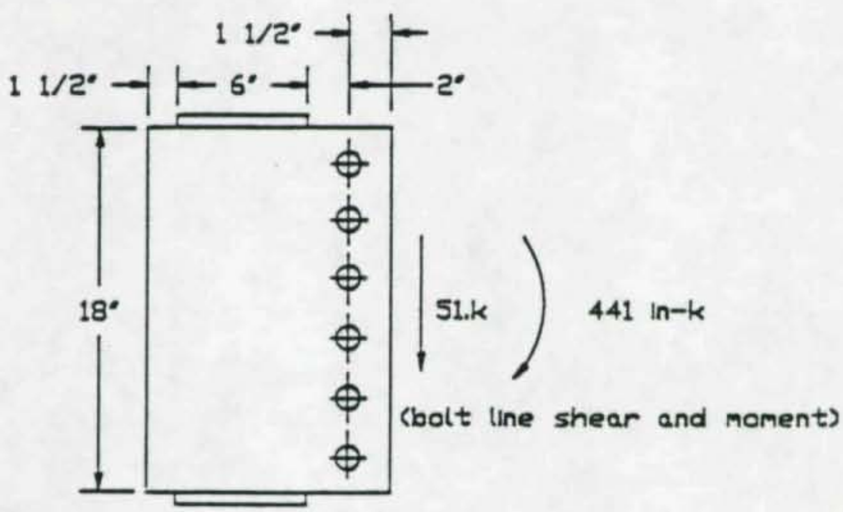
Use 5/16" fillets each side all around.

ASD DESIGN EXAMPLE 7 (Refer to Figure 1d and Example 1)

Refer to the connection design shown in Figure 1d with the same beam as in Example 1. The shear and moment at the bolt line are 51^{k} and 441 in-k., respectively. The beam frames into the weak axis of a W14x145 column.

Plate Dimensions





ASD Design Procedures

Design Steps 1 through 4 are same as for Example 1.

Step

5 Check plate stresses at edge of flange plates

$M = 441 + (51. \times 2) = 543$ in.-k

$$f_b = \frac{4 \times 543}{0.375 \times 18^2} = 17.8 \text{ ksi} < 22 \text{ ksi}$$

$$f_v = \frac{51}{0.375 \times 18} = 7.55 \text{ ksi} < 14.4 \text{ ksi}$$

6 Design welds using **AISC Manual Table XXIII**, p. 4-80.

$\ell = 18$ and $K\ell = 6"$

From the table with $K = 6/18 = 0.33$, $x = 0.0665$ so that $x\ell = 1.20"$.

From Example 1 the eccentricity was 8.65" so that

$a\ell = 8.65 + (2 + 6 + 1\text{-}1/2) - 1.20 = 16.98"$ and

$$a = \frac{16.98}{18} = 0.943$$

From the Table XXIII, $C = 0.492$

$$\text{70XX weld req'd} = \frac{51}{0.492 \times 18} = 5.76 \text{ sixteenths}$$

Use 3/16" fillets all around.

7 Use 3/8" flange plates with 3/16" fillet welds to the column.

ASD DESIGN EXAMPLE 8

Refer to connection design shown in Figure 6 with the same beam as in Example 1. The shear and moment at the bolt line are $51.^{k}$ and 441 in-k. Design the plate welds for a 12x12x1/2 tube column (F_y = 46 ksi) for a design extending the plate through the column.

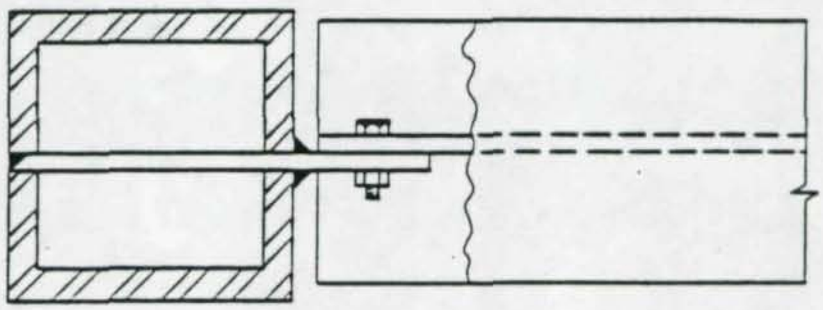
Use ASD AISC Table XIX, p. 4-76. From Example 1, e = 8.65 and ℓ = 18.
Then $a\ell$ = (8.65 + 3. + 5) = 16.65", $k\ell$ = 10.

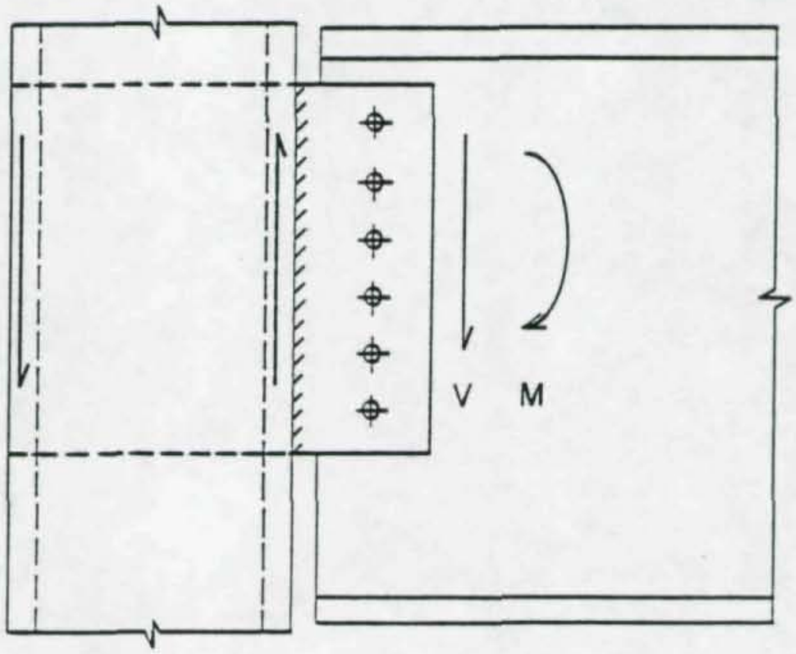
Thus $a = \frac{16.65}{18} = 0.925$ and $k = \frac{10}{18} = 0.555$.

From Table XIX, C = 0.624.

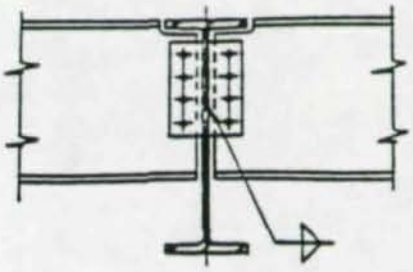
70XX weld req'd $= \frac{51.}{0.624 \times 18} = 4.54$ sixteenths

Use 5/16" fillets on the beam side of column; 3/8" bevel on the opposite side.





LRFD DESIGN EXAMPLE 1



Beam: W24 x 68, A36 Steel, $S = 154$ in^3
Span: 24', Laterally Supported
Loading: Factored Uniform Load = 159^k

LRFD Design Procedure

Step

1 Select A36 plate with $t_{plate} = 3/8"$ ($t_{web} = 0.415"$)

2 Try 3/4" A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right)/\left(\frac{3}{8}\right) = 2.0$$

$$R = \frac{159}{2} = 79.5^k$$

$$n_{req'd} = \frac{79.5^k}{15.5^k} = 6 \text{ bolts}$$

3 $$\left(\frac{e}{h}\right)_{ref} = 0.06\,\frac{L}{d} - 0.15 = 0.57$$

$$\left(\frac{e}{h}\right) = 0.57 \times \frac{6}{5} \times \left(\frac{100}{153}\right)^{0.4} = 0.58$$

With pitch = 3", h = (6 - 1) x 3 = 15"
e = 0.58 x 15 = 8.65"

4 For a = 3", $V = R = 79.5^k$
M = 79.5 x (8.65 + 3) = 927.3 in.-k

5 $$f_b = \frac{4 \times 927.3}{0.375 \times 18^2} = 30.5 \text{ ksi} < 32.4 \text{ ksi O.K.}$$

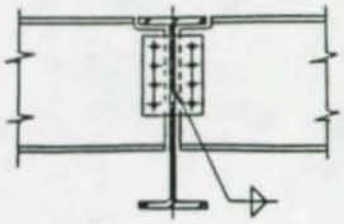
$$f_v = \frac{79.5}{0.375 \times 18} = 11.8 \text{ ksi} < 20.2 \text{ ksi O.K.}$$

6 From LRFD AISC Table XVIII p. 5-91 with $a\ell = 8.65 + 3 = 11.65$, $a = \frac{11.65}{18} = 0.65$, then C = 1.04

$$\text{70XX weld req'd} = \frac{79.5}{1.04 \times 1.0 \times 18} = 4.25 \text{ sixteenths}$$

Use 5/16" fillets each side.

LRFD DESIGN EXAMPLE 2



Beam: W24 x 62, A572 Grade 50 Steel, $S = 131\ in^3$
Span: 24', Laterally Supported
Loading: Factored Uniform Load = 176^k

LRFD Design Procedure

Step

1 Select A36 plate with $t_{plate} = 7/16"$ ($t_{web} = 0.430"$)
($t_{web} = t_{equiv} = 0.430 \times 50/36 = 0.597"$)

2 Try 7/8" A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{7}{8}\right) / \left(\frac{7}{16}\right) = 2.0$$

$$R = \frac{176}{2} = 88^k$$

$$n_{req'd} = \frac{88^k}{21.1^k} = 5 \text{ bolts}$$

3
$$\left(\frac{e}{h}\right)_{ref} = 0.06\ \frac{L}{d} - 0.15 = 0.57$$

$$\left(\frac{e}{h}\right) = 0.57 \times \frac{6}{5} \times \left(\frac{175}{131}\right)^{0.4} = 0.642$$

With pitch = 3", $h = (5 - 1) \times 3 = 12"$, and $F_y = 50$ ksi

$$e = 0.642 \times 12 \times \frac{36}{50} = 5.55"$$

4 For $a = 3"$, $V = R = 88^k$
$M = 88 \times (5.55 + 3) = 752$ in.-k

5
$$f_b = \frac{4 \times 752}{0.4375 \times 15^2} = 30.6 \text{ ksi} < 32.4 \text{ ksi O.K.}$$

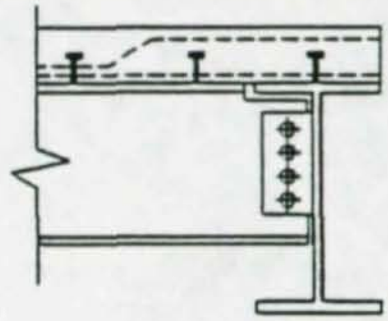
$$f_v = \frac{88}{0.4375 \times 15} = 13.4 \text{ ksi} < 20.2 \text{ ksi O.K.}$$

6
$$f_r = (30.6^2 + 13.4^2)^{1/2} = 33.4 \text{ ksi}$$

$$\text{70XX weld req'd} = \frac{33.4 \times 0.4375}{1.39} = 10.5 \text{ sixteenths}$$

Use 3/8" fillets each side.

LRFD DESIGN EXAMPLE 3



Beam: W16 x 40, A36 Steel with 4" Slab, $S_{tr} = 92.8$ in³
Span: 24', Laterally Supported
Loading: Factored Uniform Load = 100^k

LRFD Design Procedure

Step

1 Select A36 plate with $t_{plate} = 3/8"$ ($t_{web} = 0.307"$)

2 Try 3/4" A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right)/(0.307) = 2.44 > 2.0$$

$$R = \frac{100}{2} = 50^k$$

$$n_{req'd} = \frac{50^k}{15.5^k} = 4 \text{ bolts}$$

3 $$\left(\frac{e}{h}\right)_{ref} = 0.06\ \frac{L}{d} - 0.15 = 0.714$$

$$\left(\frac{e}{h}\right) = 0.714 \times \frac{4}{5} \times \left(\frac{100}{92.8}\right)^{0.4} = 0.589$$

With pitch = 3", h = (4 - 1) x 3 = 9", and F_y = 36 ksi
e = 0.589 x 9 = 5.30"

4 For a = 3", V = R = 50^k
M = 50 x (5.30 + 3) = 415 in.-k

5 $$f_b = \frac{4 \times 415}{0.375 \times 12^2} = 30.7 \text{ ksi} < 32.4 \text{ ksi}$$

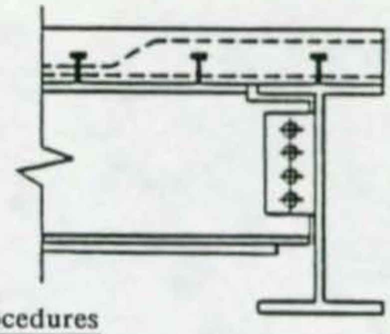
$$f_v = \frac{50}{0.375 \times 12} = 11.11 \text{ ksi} < 20.2 \text{ ksi}$$

6 $f_r = (30.7^2 + 11.11_2)^{1/2} = 32.6$ ksi

$$70XX \text{ weld req'd} = \frac{32.6 \times 0.375}{1.39} = 8.81 \text{ sixteenths}$$

Use 5/16" fillets each side.

LRFD DESIGN EXAMPLE 4



Beam: W21 x 44 with 5-1/2" x 1/2" plate
A572 Grade 50 Steel with 4" slab
$S_t = 481\ in^3$, $S_{tnp} = 406\ in^3$
(Refer to Ref. 3, p. 133)

Span: 30 ft., Laterally Supported

Loading: Uniform with $w = 200^k$

LRFD Design Procedures

Step

1 Select A36 plate with $t_{plate} = 3/8"$ [$t_{web} = 0.348"$
$t_{equiv} = 0.348 \times (50/36) = 0.483$, so A36 plate controls]

2 Try 3/4" A325x bolts either snug or fully tightened in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right) / \left(\frac{3}{8}\right) = 2.0$$

$$R = \frac{200}{2} = 100^k$$

$$n_{req'd} = \frac{100}{20.7^k} = 5 \text{ bolts}$$

3 $$\left(\frac{e}{h}\right)_{ref} = 0.06\ \frac{L}{d} - 0.15 = 0.697$$

$$\left(\frac{e}{h}\right) = 0.697 \times \left(\frac{5}{5}\right) \times \frac{100}{481})^{0.4} \times \left(\frac{406}{481}\right)^{1/2} = 0.340$$

With pitch = 3", h = (5-1) x 3 = 12"
e = 0.340 x 12 = 4.08

4 For a = 3", $V = R = 100^k$
M = 100 x (4.08 + 3) = 708 in-k

5 $$f^b = \frac{4 \times 708}{0.375 \times 15^2} = 33.5 \text{ ksi}\cdot = 32.4 \text{ ksi O.K.}$$

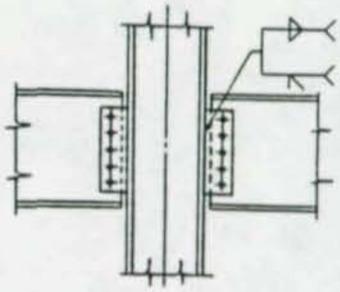
6 $$f_v = \frac{100}{0.375 \times 15} = 17.8 \text{ ksi} < 19.4 \text{ ksi}$$

7 Use LRFD AISC Table XVIII.
$\ell = 15$, $a\ell$ (708/100) = 7.08, a = 7.08/15 = 0.472, C = 1.36

$$\text{70XX weld req'd} = \frac{100}{1.36 \times 15} = 4.90$$

Use 5/16" fillets each side.

LRFD DESIGN EXAMPLE 5



Beam: W16 x 40, A36 Steel
Span: 24', Laterally Supported
Loading: Factored Uniform Load = 90^k

LRFD Design Procedure

Step

1 Select A36 plate with $t_{plate} = 5/16"$ ($t_{web} = 0.305"$)

2 Try 7/8" A307N bolts in standard holes

$$R = \frac{90}{2} = 45^k$$

$$n_{req'd} = \frac{45}{9.7^k} = 5 \text{ bolts}$$

3 For 3" pitch, h = 12"

$$\Delta_{top\ bolt} = \frac{W\ell^2}{24EI} \times \frac{h}{2} = 90 \times \frac{(24 \times 12)^2}{24 \times 30 \times 10^3 \times 518} \times \frac{12}{2} = 0.12 \simeq 0.10" \text{ say O.K.}$$

$$e = \left(\frac{5 \times 12}{384}\right)\left(\frac{20 \times 12}{16}\right) = 2.34$$

4 For a = 3", V = R = 45

$$M = 45 \times (2.34 + 3) = 240 \text{ in.-k}$$

5
$$f_b = \frac{6 \times 240}{0.316 \times 15^2} = 20.2 \text{ ksi} < 32.4 \text{ ksi}$$

$$f_v = \frac{45}{0.316 \times 15} = 9.60 \text{ ksi} < 20.2 \text{ ksi}$$

6
$$f_r = (11.86^2 + 5.55^2)^{1/2} = 22.4 \text{ ksi}$$

$$\text{70XX weld req'd} = \frac{22.4 \times 0.316}{1.39} = 5.08 \text{ sixteenths}$$

Use 3/16" fillets each side.

LRFD DESIGN EXAMPLE 6 (Refer to Figure 1b and Example 1)

Beam: W24 x 68, A36 Steel, S = 154 in^3
Span: 24', Laterally Supported
Loading: Factored Uniform Load = 159^k
Girder: W30 x 132

LRFD Design Procedure

Step

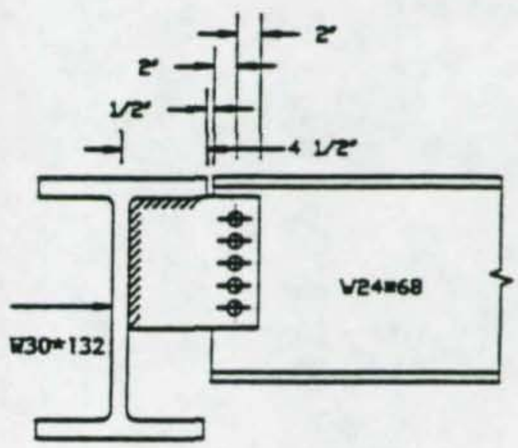
1 Select A36 plate with $t_{plate} = 3/8"$ ($t_{web} = 0.416"$)

2 Try 3/4" A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right)/\left(\frac{3}{8}\right) = 2.0$$

$$R = \frac{159}{2} = 79.6^k$$

$$n_{req'd} = \frac{79.6^k}{15.5^k} = 6 \text{ bolts}$$



3 $$\left(\frac{e}{h}\right)_{ref} = 0.06\ \frac{L}{d} - 0.15 = 0.57$$

$$\left(\frac{e}{h}\right) = 0.57 \times \frac{6}{5} \times \left(\frac{100}{153}\right)^{0.4} = 0.577$$

With pitch = 3", h = (6 - 1) x 3 = 15"
e = 0.576 x 15 = 8.65"

4 Allow 1/2" clearance between girder and beam flanges. With 2" edge distance in plate and beam web, and 1-1/2" end distances, use 8-3/4 x 3/8 x 1'-6" plate.

5 Check plate stresses at end of beam
M = 79.5 x (8.65 + 2.0) = 847 in.-k

$$f_b = \frac{4 \times 847}{0.375 \times 18^2} = 27.9 \text{ ksi} < 32.4 \text{ ksi}$$

$$f_v = \frac{79.5}{0.375 \times 18} = 11.8 \text{ ksi} < 20.2 \text{ ksi}$$

6 From LRFD AISC Table XXIV with $\ell = 18"$ and $K\ell = 4.5"$, then x = 0.026
Therefore $x\ell = 0.468$
Now $a\ell = 8.65 + 2 + 1/2 + 4.5 - 0.468 = 15.2"$, so that a = 0.84
From Table XXIV p. 5-109; C = 0.572

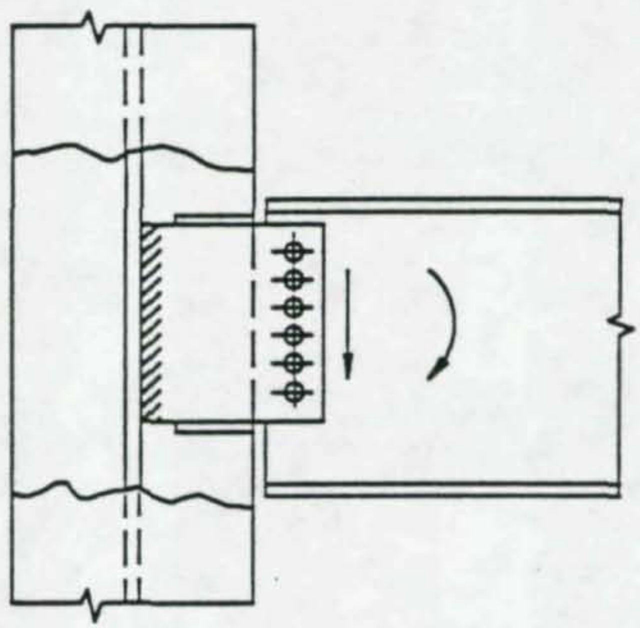
$$\text{70XX weld req'd} = \frac{79.5}{1 \times 0.572 \times 18} = 7.72 \text{ sixteenths}$$

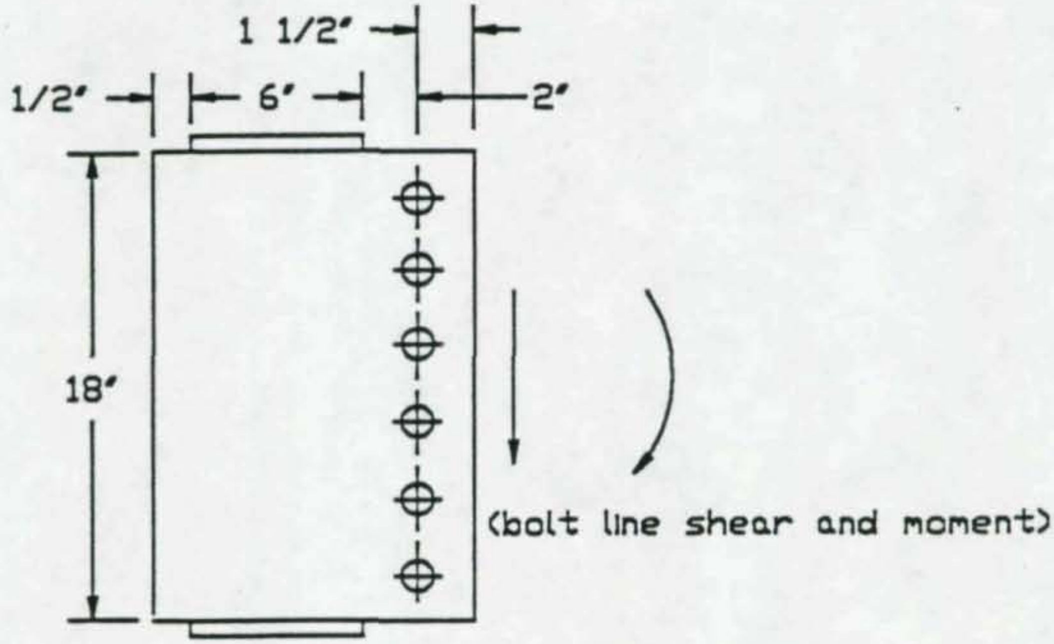
Use 1/4" fillets each side all around.

LRFD DESIGN EXAMPLE 7

Refer to the connection design shown in Figure 1d with the same beam as in Example 1. The shear and moment at the bolt line are 79.6^k and 688 in.-k., respectively. The beam frames into the weak axis of a W14x145 column.

Plate Dimensions





LRFD Design Procedures

Design Steps 1 through 4 are the same as for Example 1.

Step

5 Check plate stresses at edge of flange plates

$M = 688 + (79.6 \times 2) = 847.$ in.-k

$$f_b = \frac{4 \times 847}{0.375 \times 18^2} = 27.9 \text{ ksi} < 32.4 \text{ ksi}$$

$$f_v = \frac{79.6}{0.375 \times 18} = 11.8 \text{ ksi} < 20.2 \text{ ksi}$$

6 Design welds using **AISC Manual Table XXII**, p. 5-103, $\ell = 18$ and $K\ell = 6"$

From the table with $K = 6/18 = 0.33$, $x = 0.0665$ so that $x\ell = 1.20"$.

From Example 1 the eccentricity was 8.65" so that

$a\ell = 8.65 + (2 + 6 + 1.5) - 1.20 = 16.95"$ and

$$a = \frac{16.95}{18} = 0.94$$

From the Table XXII, $C = 0.80$

$$\text{70XX weld req'd} = \frac{79.6}{0.80 \times 18} = 5.53 \text{ sixteenths}$$

Use 3/16" fillets all around.

7 Use 3/8" flange plates with 3/16" fillet welds to the column.

LRFD DESIGN EXAMPLE 8

Refer to connection design shown in Figure 6 with the same beam as in Example 1. The shear and moment at the bolt line are 79.5^{k} and 688 in-k. Design the plate welds for a 12x12x1/2 tube column (F_y = 46 ksi) for a design extending the plate through the column.

Use ASD AISC Table XIX, p. 4-76. From Example 1, e = 8.65 and ℓ = 18. Then $a\ell$ = (8.65 + 3. + 5) = 16.65", $k\ell$ = 10.

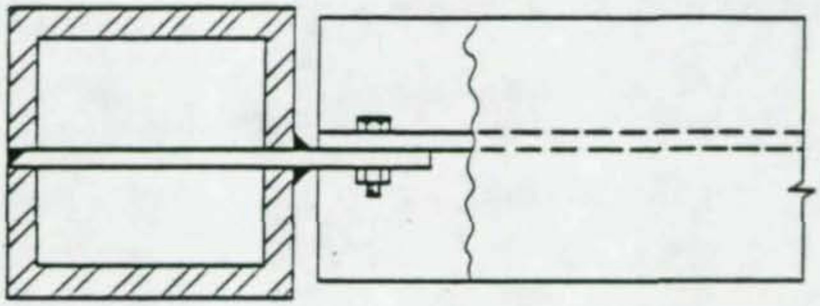
Thus $a = \frac{16.65}{18} = 0.925$ and $k = \frac{10}{18} = 0.555$.

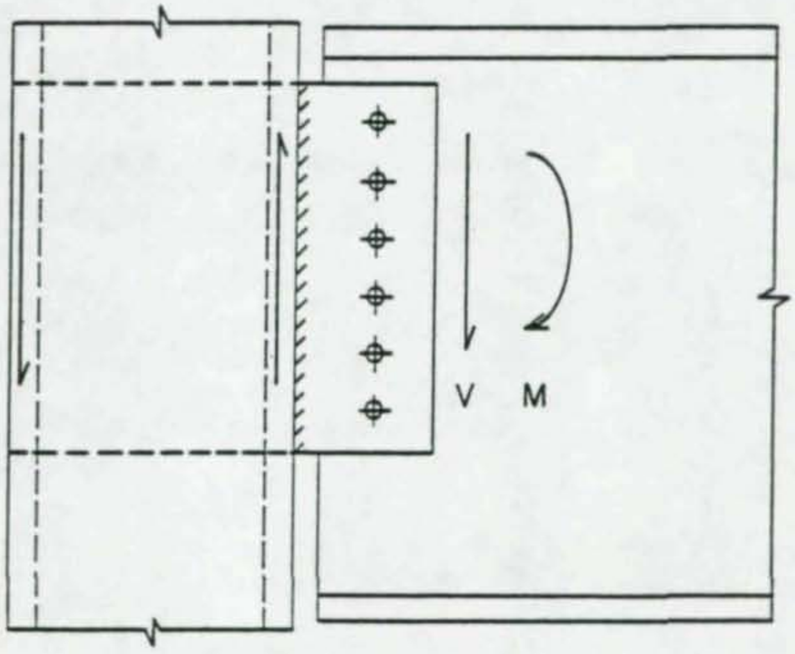
From Table XVIII, C = 1.01

70XX weld req'd $= \frac{79.5}{1.1 \times 18} = 4.38$ sixteenths

Use 5/16" fillets on the beam side of column; 3/8" bevel on the opposite side.

Alternatively, extend plate and use fillet welds on both sides of the column.





REFERENCES

1. Richard, R. M, P. E. Gillett, J. D. Kriegh and B. A. Lewis, "The Analysis and Design of Single Plate Framing Connections," *Engineering Journal*, AISC, Vol. 7, No. 2, Second Quarter, 1980.

2. Richard, R. M., J. D. Kriegh and D. E. Hormby, "Design of Single Plate Framing Connections with A307 Bolts," *Engineering Journal*, AISC, Vol. 19, No. 4, Fourth Quarter, 1982.

3. Hormby, D. E., R. M. Richard, and J. D. Kriegh, "Single Plate Framing Connections with Grade 50 Steel and Composite Construction," *Engineering Journal*, AISC, Vol. 21, No. 3, Third Quarter, 1984.

4. *American Institute of Steel Construction*, "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," Chicago, Illinois, November 1978.

5. *American Institute of Steel Construction*, "Steel Connections%/Details and Relative Costs," 1981.

6. Lewitt, C. W., E. Chesson, Jr., and W. H. Munse, "Restraint Characteristics of Flexible Riveted and Bolted Beam-To-Column Connections," *Engineering Experiment Station Bulletin 500*, College of Engineering, University of Illinois, 1969.

7. "Discussion of Reference 2," *Engineering Journal*, AISC, Vol. 22, No. 1, First Quarter, 1985.

8. Young, N. W. and R. O. Disque, "Design Aids for Single Plate Framing Connections," *Engineering Journal*, AISC, Vol. 18, No. 4, Fourth Quarter, 1981.

9. Lorenz, R. F. and F. W. Stockwell, "Concrete Slab Stresses in Partial Composite Beams and Girders," *Engineering Journal*, AISC, Vol. 21, No. 3, Third Quarter, 1984.

10. Abolitz, A. L. and M. E. Warner, "Bending Under Seated Connections," *Engineering Journal*, AISC, January 1965.

11. Salmon, C. G., and J. E. Johnson, *Steel Structures, Design and Behavior*, Second Edition, Harper and Row, 1980.

12. Stockwell, Frank W., Jr., "Yield Line Analysis of Column Webs with Welded Beam Connections," *Engineering Journal*, AISC, First Quarter, 1974.

13. Kapp, Richard H., "Yield Line Analysis of a Web Connection in Direct Tension," *Engineering Journal*, AISC, Second Quarter, 1974.

14. Elsalti, Maher KH., Simplified Single Plate Connection Designs, Master of Science in Civil Engineering Thesis, Department of Civil Engineering and Engineering Mechanics, University of Arizona, Nov. 1988. Directed by Professor R. M. Richard.

APPENDIX A

SIMPLIFIED DESIGN PROCEDURE
FOR SINGLE PLATE FRAMING CONNECTIONS

APPENDIX A

SIMPLIFIED DESIGN PROCEDURE FOR SINGLE PLATE FRAMING CONNECTIONS

A.1 Design Criteria for the Plate: (same as in chapter 2)

A.1.1 Use ASTM A36 steel plate for ductility with a single row of bolts.

A.1.2 For either snug tight or fully tightened high-strength bolts in standard round holes, design ductility into the connection by providing the following geometric properties:

$\frac{L_H}{D} > 2$ Required to prevent tension tearing.

$\frac{D}{t} > 2$ to prevent the bolt shear mode of failure for A325 bolts.

$\frac{D}{t} > 1.5$ Required to prevent the bolt shear mode of failure for A490 bolts.

where:

L_H = distance from the bolt center line to the edge plate and/or beam web.

D = bolt diameter.

t = plate or beam thickness, whichever is smaller.

or refer to Table [3], [4].

If beam is Grade 50, $t_{webequiv} = t_{web} \times 50/36$

A.1.3 If the holes are slotted, constraints in A.1.2 are not required, so use standard edge distance (AISC ASD Specification Table 1.16.5.1 and LRFD Table J3.7).[1]

A.1.4 If the beam web thickness controls in A.1.2 above, and the beam is coped, the block shear mode of failure should be checked. Generally, this failure mode will not control the design because the bolts are in single shear, but may if the connection depth is less than one-half the beam depth.[1]

A.1.5 If A307 bolts are used in standard round holes, limit the maximum bolt distortion to less than 0.10 inches as shown in Figure (4). That is, maintain $\Delta_{topbolt} = \phi_{simplebeam} \times (h/2) < 0.10$ in., where $\phi_{simplebeam} = (wL^2)/(24EI)$ for uniformly distributed loads.[1]

A.1.6 If snug tight A325 and A490 bolts are used in either short or long slotted holes, the center holes of the bolt pattern need not be slotted. This detail can be useful for alignment of the structure during erection.[1]

A.2 Design Criteria for the Bolts (same as chapter 2)

A.2.1 Compute the number of bolts required by dividing the beam shear by the allowable bolt load.

A.2.2 Use only a single row of bolts.

A.3 Design Criteria for the Weldment (same as chapter 2)

A.3.1 If fully tightened A325 or A490 bolts are used in either round or slotted holes, or if snug tight A325 or A490 bolts are used in standard holes, compute the bolt line moment as follows:

1. Select plate thickness ± 1/16 in. of supported beam web.

$$t_{plate} = t_{web} \pm 1/16$$

2. Compute number of bolts required based upon beam shear and allowable bolt loads.

3. Enter the Bolt Line Moment Table A-1 (ASD) or A-2 (LRFD) with diameter and number of bolts and web thickness to find M_{bolt}

   FOR pitch different than 3. in., multiply M_{bolt} by the ratio, **pitch/3**.

4. If the beam is not uniformly loaded, increase the M_{bolt} or the eccentricity using the coefficient from Table A-3; that is,

$$(M_{bolt})_{conc} = (M_{bolt})_{uniform} \times \text{coefficient}$$

5. Compute the moment at the weld line, M_{conn}, as follows:

$$M_{conn} = M_{bolt} + R \times a$$

   where

   R = beam shear force.

   a = distance from the bolt line to the center of gravity of the weldment.

6. Check the plate normal and shear stresses;

$$f_b = \frac{4 \times M_{conn}}{t\, b^2} < 22.0 \text{ Ksi (ASD)}$$

$$= \frac{4 \times M_{conn}}{t\, b^2} < 32.4 \text{ Ksi (LRFD)}$$

$$f_v = \frac{R}{b\, t} < 14.4 \text{ Ksi (ASD)}$$

$$= \frac{R}{b\, t} < 19.4 \text{ Ksi (LRFD)}$$

where t and b are the plate thickness and depth, respectively.

7. Design the weldment based upon the resultant of the normal and shear stresses from step 7:

$$f_r = \left(f_b^2 + f_v^2\right)^{0.5}$$

70XX weld req'd

$$= \frac{f_r \times t}{2 \times 0.93} \text{ sixteenths (ASD)}$$

$$= \frac{f_{rx}\, t}{2 \times 1.39} \text{ sixteenths (LRFD)}$$

which gives the size of fillet welds on each side of the plate.

8. Alternatively, the weld may be designed using the ASIC Manual eccentrically loaded weld group design aids.

A.3.2 If A307 bolts in standard holes or snug tight A325 or A490 bolts in slotted holes are used, compute the beam eccentricity e as shown in Figure (5) as follows:

$$e = \left(\frac{n \times h}{384}\right) \times \left(\frac{L}{d}\right)$$

where

n = number of bolts

h = depth of bolt pattern

L = length of beam

d = depth of beam (total depth of composite beams)

Table A-4 provides the eccentricity depending on number of bolts and L/d ratio (the span divided by the beam depth). With e known calculate the bolt line moment and complete the design starting with Step 4 above.

A.3.3 For cases included in **A.3.2**, if the beam is of Grade 50 steel, reduce the eccentricity by the ratio of steel strength; that is,

$$e_{50} = e_{36} \times (36/50)$$

A.3.4 For cases included in **A.3.2**, the connection moment at the weld line as shown in Figure (5) is equal to

$$M_{conn} = R \times (a+e)$$

where

R = beam reaction

a = distance from the bolt line to C.G. of the weldment as in Figure (3).

e = eccentricity from **A.3.2**

Check the plate normal and shear stresses;

$$f_b = \frac{6 \times M_{conn}}{t\ b^2} < 22.0 \text{ Ksi (ASD)}$$

$$= \frac{6 \times M_{conn}}{t\ b^2} < 32.4 \text{ Ksi (LRFD)}$$

$$f_v = \frac{R}{b\ t} < 14.4 \text{ Ksi (ASD)}$$

$$= \frac{R}{b\ t} < 19.4 \text{ Ksi (LRFD)}$$

where t and b are the plate thickness and depth, respectively.

A.3.5 If the beam is not uniformly loaded, increase the M_{bolt} or the eccentricity using the coefficient from Table A-3 which is from Reference 1; that is,

$$(M_{bolt})_{conc} = (M_{bolt})_{uniform} \times \text{coefficient}$$

$$\text{or } e_{conc} = e_{uniform} \times \text{coefficient}$$

A.4 Beam L/d Limits and End Rotation

A.4.1 Noncomposite Beams

To insure connection ductility by avoiding bolt shear and tension tearing of the plate or beam web when using high-strength bolts, in standard holes, it recommended that beam end rotations should be limited to a rotation that causes 0.2-in. deformation at the outer bolts, as shown in Figure (4), at 1.5 times the service load for **ASD** or at the factored load for **LRFD**.

A.4.2 **Simple Beam End Rotation Based on the L/360 Criterion:**

The end rotation for simple beam is given by the formula

$$\phi = (wL^3)/(24EI)$$

where

w = uniform load per unit length

L = beam span

E = Young's Modulus

I = Beam Moment of Inertia (Transformed for Composite Beams)

The midspan deflection is

$\delta = (5/384)(wL^4)/(EI) = (120/384)\ L$

with $\delta = L/360$, then $\phi = 0.00889$

So for connections with pitch of 3 in., at 1.5 times the service load,

$$\Delta_{topbolt} = 1.5 \times \phi \times h/2 = 1.5 \times \phi \times (N - 1) \times 3/2$$

$$\Delta_{topbolt} = 0.02\ (N - 1) \leq\ 0.20 \text{ in.}$$

where: N = number of bolts

Thus for connections with 10 or less bolts this criterion is satisfied.

TABLE A-1 ASD BOLT LINE MOMENT IN INCH-KIPS

For 3/4, 7/8, 1., and 1 1/8 in. Diameter High Strength Bolts
(a) Fully Tightened in Standard Round or Slotted Holes
(b) Snug Tight in Standard Round Holes
and for Beam Steel Grade 36 and 50

NO. OF BOLTS	BEAM WEB THICKNESS t_{web} in.							
NB	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16
3	80	90	110	130	160	—	—	—
4	110	150	190	240	275	325	375	—
5	—	200	275	360	450	525	600	700
6	—	—	400	500	650	750	850	950
7	—	—	500	725	900	1050	1200	1350
8	—	—	—	900	1050	1250	1500	1800
9	—	—	—	—	1250	1600	1900	2100
10	—	—	—	—	—	2000	2300	2600

TABLE A-2 LRFD BOLT LINE MOMENT INCH-KIPS

For 3/4, 7/8, 1, and 1 1/8 in. diameter High Strength Bolts in
(a) Fully Tightened in Standard Round or Slotted Holes
(b) Snug Tight in Standard Round Holes
and for Beam Steel Grade 36 and 50

NO. OF BOLTS	BEAM WEB THICKNESS t_{web} in.							
NB	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16
3	115	130	160	185	230	—	—	—
4	160	215	270	350	400	465	540	—
5	—	290	400	515	650	750	860	1000
6	—	—	575	715	930	1075	1225	1375
7	—	—	715	1050	1300	1500	1725	1950
8	—	—	—	1300	1500	1800	2150	2575
9	—	—	—	—	1800	2300	2725	3000
10	—	—	—	—	—	2900	3300	3725

TABLE A-3 CONCENTRATED LOAD ECCENTRICITY COEFFICIENTS

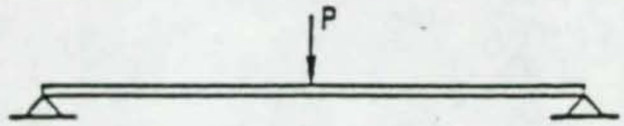
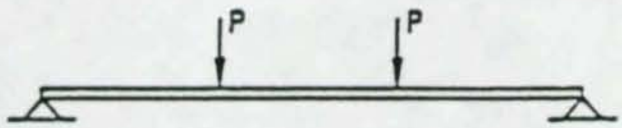
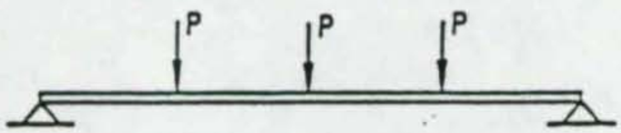
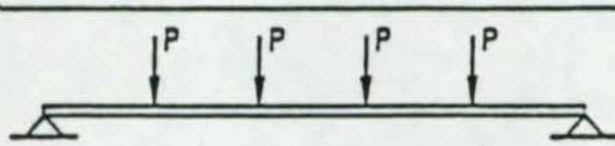
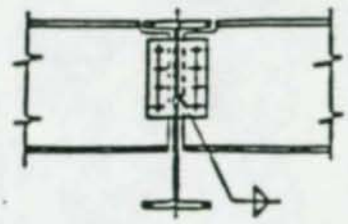
TYPE OF LOADING EQUAL LOADS, EQUAL SPACES	ECCENTRICITY COEFFICIENT
	2.00
	1.33
	1.33
	1.20

TABLE A-4. Bolt Line Eccentricities in Inches For A307 Bolts And Snug Tight High Strength Bolts in Slotted Holes For 3 in. Pitch

NO. OF BOLTS NB	L/d ratio											
	8	10	12	14	16	18	20	22	24	26	28	30
2	0.13	0.16	0.19	0.22	0.25	0.28	0.31	0.34	0.38	0.41	0.44	0.47
3	0.38	0.47	0.56	0.66	0.75	0.84	0.94	1.03	1.13	1.22	1.31	1.41
4	0.75	0.94	1.13	1.31	1.50	1.69	1.88	2.06	2.25	2.44	2.63	2.81
5	1.25	1.56	1.88	2.19	2.50	2.81	3.13	3.44	3.75	4.06	4.38	4.69
6	1.88	2.34	2.81	3.28	3.75	4.22	4.69	5.16	5.63	6.09	6.56	7.03
7	2.63	3.28	3.94	4.59	5.25	5.91	6.56	7.22	7.88	8.53	9.19	9.84
8	3.50	4.38	5.25	6.13	7.00	7.88	8.75	9.63	10.50	11.38	12.25	13.13
9	4.50	5.63	6.75	7.88	9.00	10.13	11.25	12.38	13.50	14.63	15.75	16.88
10	5.63	7.03	8.44	9.84	11.25	12.66	14.06	15.47	16.88	18.28	19.69	21.09

PLES



Beam: W24 x 68, A36 Steel,

Span: 24 ft, Laterally Supported

Loading: Uniform load with W = 102 K

Simplified Design Procedure

Step

1 Steel A36 plate with $t_{plate} = 3/8$ in. ($t_{web} = 0.415 = 7/16$) in.

2 Try 3/4 in. A325N bolts (either snug or fully tightened) in standard holes

$\frac{D}{t} = (3/4)/(3/8) = 2.0$

$R = 102/2 = 51$ K

$NB_{req'd} = 51^k/9.28^k = 6$ bolts

3 With pitch = 3 in., and D= 3/4 Enter Table [2]

with $t = 7/16$ in. and NB=6 find $M_{bolt} = 500$ K-in.

4 $M_{conn} = 500 + (51 \times 3) = 653$ K-in

5 $f_b = 4 \times 653/(0.375 \times 18^2) = 21.5 < 22.$ ksi

$f_v = 51/(0.375 \times 18) = 7.56 < 14.4$ ksi

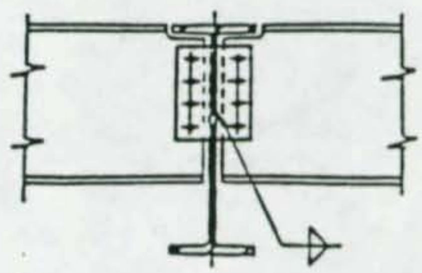
6 Use ASD AISC Table XIX.

l=18 , al = (653 / 51) = 12.8 , a = 12.8 / 18 = 0.71 , C = 0.578

70XX weld req'd = 51 / (0.578 x 18) = 4.9 sixteenths

Use 5/16 in. fillets each side.

ASD Design Example 2



Beam: W24 x 62, A572 Grade 50 Steel,

Span: 24 ft, Laterally Supported

Loading: Uniform load with W = 120 K

Simplified Design Procedure

Step

1 Steel A36 plate with $t_{plate} = 7/16$ in. $(t_{web} = 0.430 = 7/16)$

2 Try 7/8 in. A325N bolts (either snug or fully tightened)

in standard holes

$\frac{D}{t} = (7/8)/(7/16) = 2.0$

$R = 120/2 = 60$ K

$NB_{req'd} = 60^k/12.63^k = 5$ bolts

3 With pitch = 3 in. and D = 7/8 in. Enter Table [2]

with $t = 7/16$ in. and NB = 5 find $M_{bolt} = 350$ K-in.

4 $M_{conn} = 350 + (60 \times 3) = 530$ K-in

5 $f_b = 4 \times 530/(0.4375 \times 15.^2) = 21.54 < 22.$ ksi

$f_v = 60/(0.4375 \times 15) = 9.14 < 14.4$ ksi

6 Use ASD AISC Table XIX.

l = 15 , al = (530 / 60) = 8.83 , then

a = 8.3/15 = 0.589 , C = 0.686

70XX weld req'd = 60 / (0.686 x 15) = 5.83 sixteenths

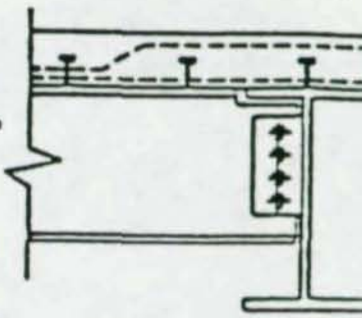
Use 3/8 in. fillets each side.

ASD Design Example 3

Beam: W16 x 40, A36 Steel, with 4 in. slab,

Span: 24 ft, Laterally Supported

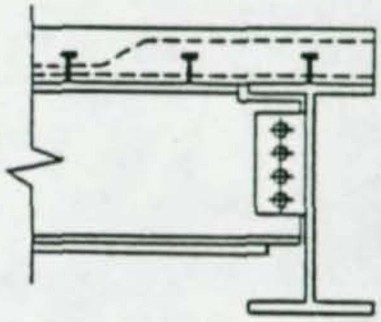
Loading: Uniform load with W = 61.9 K



Simplified Design Procedure

Step	
1	Steel A36 plate with $t_{plate} = 5/16$ in. $(t_{web} = 0.307 = 5/16)$
2	Try 3/4 in. A325N bolts (either snug or fully tightened) in standard holes
	$\frac{D}{t} = (3/4)/(0.307) = 2.4$
	$R = 61.9/2 = 30.9$ K
	$NB_{req'd} = 30.9^k/9.28^k = 4$ bolts
3	With pitch = 3 in. and D = 3/4 in. Enter Table [2] with $t = 5/16$ in. and NB = 4 find $M_{bolt} = 150$ K-in.
4	$M_{conn} = 150 + (30.9 \times 3) = 242.7$ K-in
5	$f_b = 4 \times 242.7/(0.3215 \times 12^2) = 21.6 > 22.$ Ksi
	$f_v = 30.9/(0.3125 \times 12) = 8.24 < 14.4$ ksi
6	Use ASD AISC Table XIX.
	l = 12 , al = (242.7/30.9) = 7.854
	a = 7.854/12 = 0.655 , C = 0.625
	70XX weld req'd = 30.9 / (0.625 x 12) = 4.12 sixteenths
	Use 1/4 in. fillets each side.

ASD Design Example 4



Beam: W21 x 44, with 5-1/2 in. x 1/2 in. plate,
A572 Grade Steel, with 4 in. slab,

Span: 30 ft, Laterally Supported

Loading: Uniform load with W = 130 K

Simplified Design Procedure

Step

1 Steel A36 plate with $t_{plate} = 3/8$ in. $(t_{web} = 0.348 = 3/8)$in.

$t_{webequiv} = 0.348 \times (50/36) = 0.483$ in.

2 Try 3/4 in. A490N bolts (either snug or fully tightened) in standard holes

$\frac{D}{t} = (3/4)/(3/8) = 2.0$

$R = 130/2 = 65$ K

$NB_{req'd} = 65^k/12.4^k = 6$ bolts

3 With pitch = 3 in. and D = 3/4 in. Enter Table [2] with $t = 3/8$ in. and NB = 6 find $M_{bolt} = 400$ K-in.

4 $M_{conn} = 400 + (65 \times 3) = 595$ K-in

5 $f_b = (4 \times 595)/(0.375 \times 18^2) = 19.4 < 22.$ ksi

$f_v = 65/(0.375 \times 18) = 9.63 < 14.4$ ksi

6 Use ASD AISC Table XIX , with l = 18 and,

al = (595/65) = 9.15 , a = (9.15/18) = 0.509 , C = 0.777

70XX weld req'd = 65 / (0.777 x 18) = 4.65 sixteenths

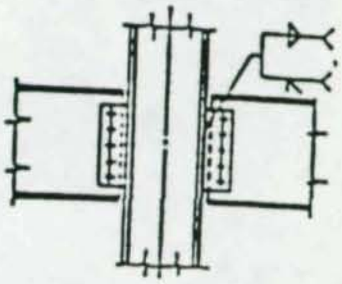
Use 5/16 in. fillets each side.

ASD Design Example 5

Beam: W16 x 40, A36 Steel,

Span: 24 ft, Laterally Supported

Loading: Uniform load with W = 52 K



Simplified Design Procedure

Step

1 Steel A36 plate with $t_{plate} = 5/16$ in. ($t_{web} = 0.305 = 5/16$) in.

2 Try 7/8 in. A307 bolts in standard holes

$R = 52/2 = 26$ K

$NB_{req'd} = 26^k/6^k = 5$ bolts

3 For pitch = 3 in., h = 12 in. At 1.5 times working load.

$\Delta_{topbolt} = 1.5 \times (Wl^2/24EI) \times (h/2)$

$= 1.5 \times \{52 \times (24 \times 12)^2/(24 \times 30 \times 10^3 \times 518)\} \times (12/2)$

$= 0.103 \approx 0.1 in.$, say OK

L/d = (24 x 12) / 16 = 18 , and NB = 5

Enter Table [7], find $e = 2.81$

4 $M_{conn} = 26 \times (2.81 + 3) = 151$ K-in

5 $f_b = 6 \times 151/(0.316 \times 15^2) = 12.75 < 22.$ ksi

$f_v = 26/(0.316 \times 15) = 5.55 < 14.4$ ksi

6 Use ASD AISC Table XIX.

l=15 , al = (2.81 + 3) = 5.81 , a = 5.81/15 = 0.387 , C = 0.964

70XX weld req'd = 26 / (0.964 x 15) = 1.8 sixteenths

Use 1/8 in. fillets each side.

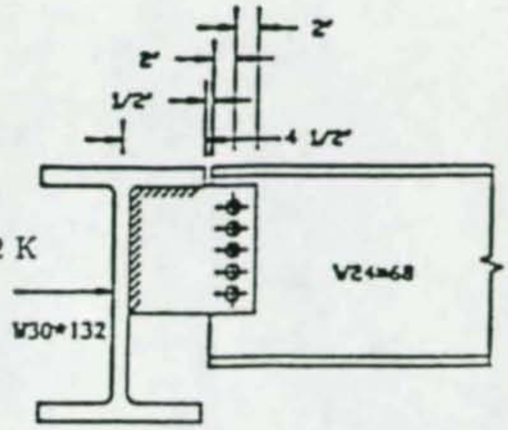
ASD Design Example 6

Beam: W24 x 68, A36 Steel,

Span: 24 ft, Laterally Supported

Loading: Uniform load with W = 102 K

Girder: W30 x 132



Simplified Design Procedure

Step

Design Step 1 through 3 are same as for Example 1. ASD

4 Allow 1/2 in. clearance between girder and beam flanges.

With 2in. edge distance in plate and beam web,

and 1-1/2 in. end distances, use 8-3/4 x 3/8 x 18 in. plate

$M_{conn} = 500 + (51 \times 2) = 602$ K-in

5 $f_b = 4 \times 602/(0.375 \times 18^2) = 19.82 < 22.$ ksi

$f_v = 51/(0.375 \times 18) = 7.56 < 14.4$ ksi

6 Use ASD AISC Table XXV .

l = 18 in. and Kl = 4.5 in.,

then x = 0.025 , therefore xl = 0.45

al = (602/51) + 1/2 + 6 - 0.45 = 17.85

a = 17.85/18 = 0.992 , C = 0.289

70XX weld req'd = 51 / (0.289 x 18) = 9.79 sixteenths

Use 5/16 in. fillets each side all around.

ASD Design Example 7

Beam: W24 x 68, A36 Steel,

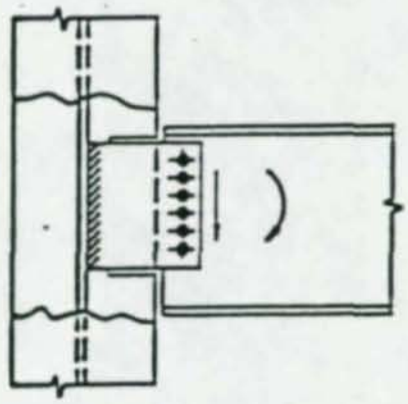
Span: 24 ft, Laterally Supported

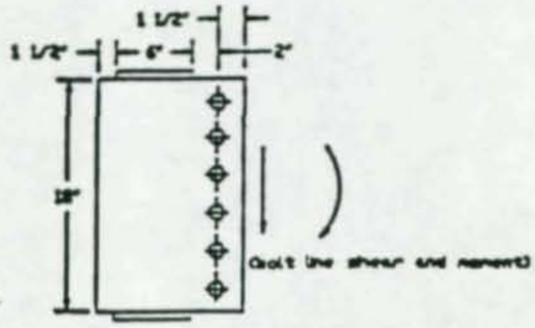
Loading: Uniform load with W = 102 K

Column: W14 x 145

The beam frames into the weak axis of the column.

Plate Dimensions:





Simplified Design Procedure

Step

Design Step 1 through 3 are same as for Example 1. ASD

4 $M_{conn} = 500 + (51 \times 2) = 602$ K-in

5 $f_b = 4 \times 602/(0.375 \times 18^2) = 19.82 < 22.$ ksi

$f_v = 51/(0.375 \times 18) = 7.56 < 14.4$ ksi

6 Use ASD AISC Table XXIII.

l = 18 in. and Kl = 6 in.,

From the table with K = 6/18 = 0.33 , then x = 0.0665

therefore xl = 1.2 in.

al = (602/51) + 6 + 1.5 - 1.2 = 18.104 in.

a = 18.104/18 = 1.006

From Table XXIII, C = 0.478

70XX weld req'd = 51 / (0.478 x 18) = 5.93 sixteenths

Use 3/16 in. fillets all around.

7 Use 3/8 in. flange plates with 3/16 fillet welds to the column.

ASD Design Example 8

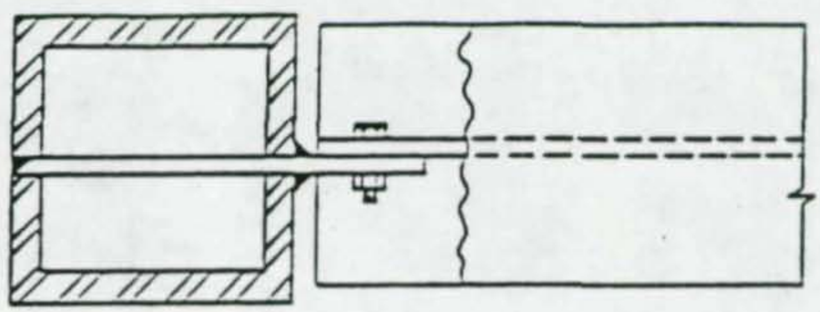
Beam: W24 x 68, A36 Steel,

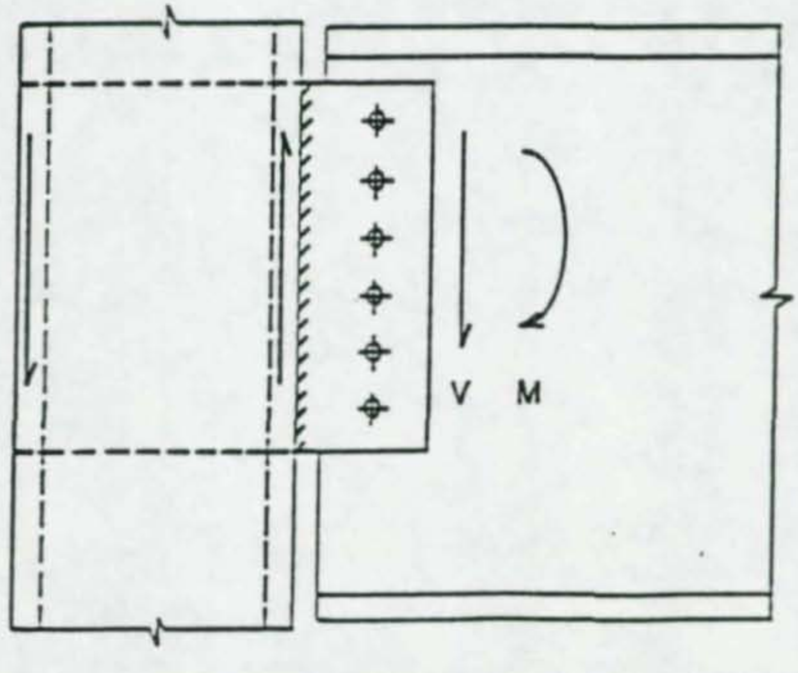
Span: 24 ft, Laterally Supported

Loading: Uniform load with W = 102 K

Column: Tube 12 x 12 x 1/2 in (F_y = 46 Ksi)

Design the plate welds for a design extending the plate through the column





Simplified Design Procedure

Step

Design Step 1 through 3 are same as for Example 1. ASD

The shear and moment at bolt line are

R = 51 K $\quad M_{bolt} = 500$ K-in

4 $\quad M_{conn} = 500 + 51 \times 3 = 653$ K-in

5 $\quad f_b = 4 \times 653/(0.375 \times 18^2) = 21.5 < 22.$ ksi

$f_v = 51/(0.375 \times 18) = 7.56 < 14.4$ ksi

6 $\quad$ Use ASD AISC Table XIX .

l = 18 in. and Kl = 12 in.

al = (653/51) + 6 = 18.8

a = 18.8/18 = 1.04 , and K = 12/18 = 0.67

From Table XIX, C = 0.626

70XX weld req'd = 51 / (0.626 x 18) = 4.53 sixteenths

Use 5/16 in. fillets on beam side of column;

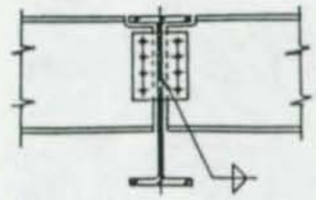
3/8 in. bevel on the opposite side.

Alternatively, extend plate and use fillet welds

on both sides of the column.

LRFD DESIGN EXAMPLE 1

Beam: W24 x 68, A36 Steel
Span: 24 ft., Laterally Supported
Loading: Factored Uniform load with W = 159 K



Simplified Design Procedure

Step

1 Steel A36 plate with $t_{plate} = 3/8$ in. ($t_{web} = 0.415 = 7/16$) in.)

2 Try 3/4 in. A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right) / \left(\frac{3}{8}\right) = 2.0$$

$$R = \frac{159}{2} = 79.5 \text{ K}$$

$$NB_{req'd} = \frac{79.5^{k}}{15.5^{k}} = 6 \text{ bolts}$$

3 With pitch = 3 in., and D = 3/4 in. Enter Table [5] with t = 7/16 in. and NB = 6 find $M_{bolt} = 715$ K-in.

4 $M_{conn} = 715 + (79.5 \times 3) = 953.5$ K-in.

5 $f_b = 4 \times \dfrac{953.3}{0.375 \times 18^2} = 31.4 < 32.4$ ksi

$$f_v = \frac{79.5}{0.375 \times 18} = 11.78 < 19.4 \text{ ksi}$$

6 Use LRFD AISC Table XVIII.

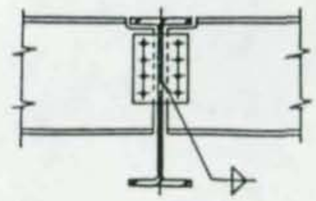
$$l = 18, \; al = \left(\frac{953.5}{79.5}\right) = 12$$

$$a = \left(\frac{12}{18}\right) = 0.67, \; C = 1.01$$

$$70XX \text{ weld req'd} = \frac{79.5}{1.01 \times 18} = 4.37 \text{ sixteenths}$$

Use 5/16 in. fillets each side.

LRFD DESIGN EXAMPLE 2



Beam: W24 x 62, A572 Grade 50 Steel
Span: 24 ft., Laterally Supported
Loading: Factored Uniform Load with W = 176 K

Simplified Design Procedure

Step

1 Steel A36 plate with $t_{plate} = 7/16"$ ($t_{web} = 0.43 = 7/16$)

2 Try 7/8" A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{7}{8}\right) / \left(\frac{7}{16}\right) = 2.0$$

$$R = 176/2 = 88 \text{ K}$$

$$NB_{req'd} = \frac{88^k}{21.1^k} = 5 \text{ bolts}$$

3 With pitch = 3 in. and D = 7/8 in. Enter Table [5] with t = 7/16 in. and NB = 5 find $M_{bolt} = 515$ K-in.

4 $M_{conn} = 515 + (88 \times 3) = 779$ K-in

5 $f_b = 4 \times \dfrac{779}{0.4375 \times 15^2} = 31.65 < 32.4$ ksi

$$f_v = \frac{88}{0.4375 \times 15} = 13.41 < 19.42 \text{ ksi}$$

6 Use LRFD AISC Table XVIII.

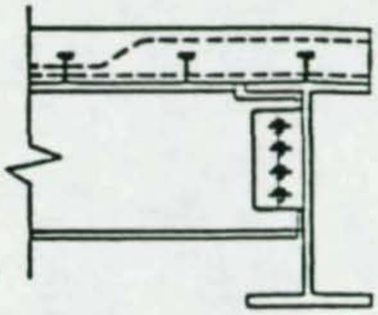
$l = 15$, $al = (779/88) = 8.85$, then

$$a = \frac{8.85}{15} = 0.59,\ C = 1.126$$

$$\text{70XX weld req'd} = \frac{88}{1.126 \times 15} = 5.21 \text{ sixteenths}$$

Use 3/8 in. fillets each side.

LRFD DESIGN EXAMPLE 3



Beam: W16 x 40, A36 Steel with 4 in. Slab
Span: 24 ft., Laterally Supported
Loading: Factored Uniform Load with W = 100 K

Simplified Design Procedure

Step

1 Steel A36 plate with $t_{plate} = 5/16$ in. ($t_{web} = 0.307 = 5/16$)

2 Try 3/4 in. A325N bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right)/(0.307) = 2.4$$

$$R = \frac{100}{2} = 50 \text{ K}$$

$$NB_{req'd} = \frac{50^k}{15.5^k} = 4 \text{ bolts}$$

3 With pitch = 3 in. and D = 3/4 in. Enter Table [5]

with t = 5/16 in. and NB = 4 find $M_{bolt} = 215$ K-in.

4 $M_{conn} = 215 + (50 \times 3) = 365$ K-in

5 $f_b = 4 \times \dfrac{365}{0.3125 \times 12^2} = 32.44 \quad 32.4$ ksi

$f_v = \dfrac{50}{0.3125 \times 12} = 13.33 < 19.4$ ksi

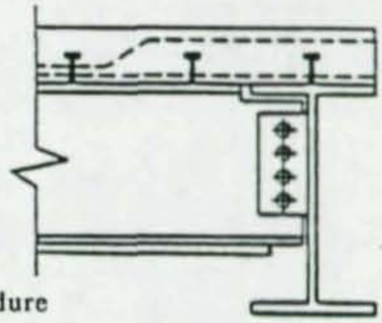
6 Use LRFD AISC Table XVIII.

l = 12, al = (365/50) = 7.3

a = (7.3/12) = 0.608, C = 1.095

70XX weld req'd = 50/(1.095 x 12) = 3.8 sixteenths

Use 1/4 in. fillets each side.



LRFD DESIGN EXAMPLE 4

Beam: W21 x 44, with 5-1/2 in. x 1/2 in. plate,
A572 Grade 50 Steel, with 4 in. Slab,
Span: 30 ft., Laterally Supported
Loading: Factored Uniform Load with W = 200 K

Simplified Design Procedure

Step

1 Steel A36 plate with $t_{plate} = t_{web} = 3/8$ in. ($t_{web} = 0.348$)

$t_{webequiv} = 0.348 \times (50/36) = 0.483$ in.

2 Try 3/4" A325X bolts (either snug or fully tightened) in standard holes

$$\frac{D}{t} = \left(\frac{3}{4}\right) / \left(\frac{3}{8}\right) = 2.0$$

$$R = \frac{200}{2} = 100$$

$$NB_{req'd} = \frac{100^k}{20.7^k} = 5 \text{ bolts}$$

3 With pitch = 3 in. and D = 3/4 in. Enter Table [5]
with t = 3/8 in. and NB = 5 find $M_{bolt} = 400$ K-in.

4 $M_{conn} = 1400 + (100 \times 3) = 700$ K-in.

5 $f_b = 4 \times \dfrac{700}{0.375 \times 15^2} = 32.2 < 32.4$ ksi

$f_v = \dfrac{100}{0.375 \times 15} = 17.78 < 19.4$ ksi

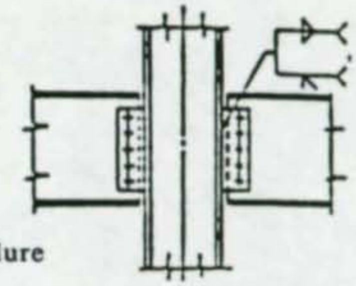
6 Use LRFD AISC Table XVIII.

$l = 15$, $al = \left(\frac{700}{100}\right) = 7$, $a = \left(\frac{7}{15}\right) = 0.467$, $C = 1.382$

70XX weld req'd = 100/(1.382 x 15) = 4.82 sixteenths

Use 5/16 in. fillets each side.

LRFD DESIGN EXAMPLE 5



Beam: W16 x 40, A36 Steel
Span: 24 ft., Laterally Supported
Loading: Uniform Load with W = 90 K

Simplified Design Procedure

Step

1 Steel A36 plate with $t_{plate} = 5/16$ in. ($t_{web} = 0.305 = 5/16$) in.

2 Try 7/8 in. A307 bolts in standard holes

$$R = \frac{90}{2} = 45 \text{ K}$$

$$NB^{req'd} = \frac{45^k}{9.7^k} = 5 \text{ bolts}$$

3 For pitch = 3 in., h = 12 in.

$$\Delta_{top\ bolt} = \frac{W\ell^2}{24EI} \times \left(\frac{h}{2}\right)$$

$$= \left\{90 \times \frac{(24 \times 12)^2}{24 \times 30 \times 10^3 \times 518}\right\} \times \left(\frac{12}{2}\right)$$

$$= 0.12 \quad 0.10 \text{ in, say O.K.}$$

$$\frac{L}{d} = \frac{24 \times 12}{16} = 18, \text{ and } NB = 5$$

Enter Table [7], find e = 2.81

4 $M_{conn} = 45 \times (2.81 + 3) = 261.5$ K-in.

5 $f_b = 6 \times \dfrac{261.5}{0.316 \times 15^2} = 22.07 < 32.4$ ksi

$$f_v = \frac{45}{0.316 \times 15} = 9.5 < 19.4 \text{ ksi}$$

6 Use LRFD AISC Table XVIII.

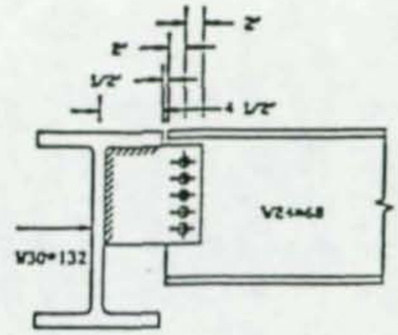
$$l = 15,\ al = (2.81 + 3) = 5.81,\ a = \frac{5.81}{15} = 0.387,\ C = 1.591$$

$$70XX \text{ weld req'd} = \frac{45}{1.591 \times 15} = 1.885 \text{ sixteenths}$$

Use 1/8 in. fillets each side.

LRFD DESIGN EXAMPLE 6

Beam: W24 x 68, A36 Steel
Span: 24 ft, Laterally Supported
Loading: Uniform Load with W = 159 K
Girder: W30 x 132



Simplified Design Procedure

Step

Design Step 1 through 3 are same as for Example 1, LRFD

4 Allow 1/2 in. clearance between girder and beam flanges. With 2 in. edge distance in plate and beam web, and 1-1/2 in. end distances, use 8-3/4 x 3/8 x 18 in. plate.
$M_{conn} = 715 + (79.5 \times 2) = 874$ K-in.

5 $f_b = 4 \times \dfrac{874}{0.375 \times 18^2} = 28.78 < 32.4$ ksi

$f_v = \dfrac{79.5}{0.375 \times 18} = 11.8 < 19.4$ ksi

6 Use LRFD AISC Table XXIV.
$l = 18$ in. and $Kl = 4.5$ in.,
then $x = 0.026$, therefore $xl = 0.468$

$al = \left[\dfrac{874.2}{79.5}\right] + \dfrac{1}{2} + 6 - 0.468 = 15.548$ in.

$a = \dfrac{15.548}{18} = 0.864$, $C = 0.563$

70XX weld req'd = $\dfrac{79.5}{0.563 \times 18} = 7.85$ sixteenths

Use 1/4 in. fillets each side all around.

LRFD Design Example 7

Beam: W24 x 68, A36 Steel,

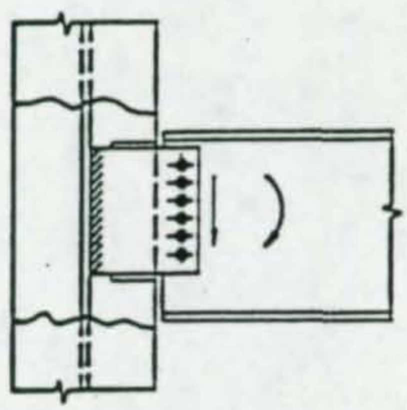
Span: 24 ft, Laterally Supported

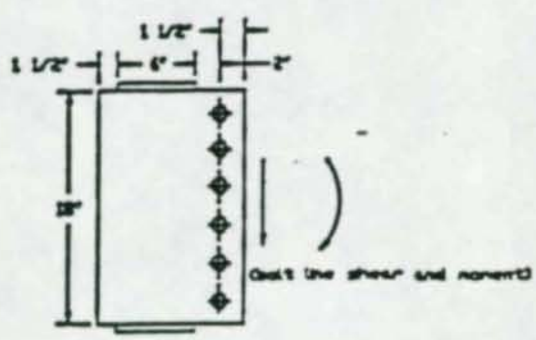
Loading: Uniform load with W = 159 K

Column: W14 x 145

The beam frames into the weak axis of the column.

Plate Dimensions:





Simplified Design Procedure

Step

Design Step 1 through 3 are same as for Example 1, LRFD

4 $M_{conn} = 715 + (79.5 \times 2) = 874$ K-in.

5 $f_b = 4 \times \dfrac{874}{0.375 \times 18^2} = 28.8 < 32.4$ ksi

$f_v = \dfrac{79.5}{0.375 \times 18} = 11.8 < 19.4$ ksi

6 Use LRFD AISC Table XXIII.

$l = 18$ in. and $Kl = 6$ in.,

From the table with $K = \dfrac{6}{18} = 0.33$, then $x = 0.0665$

therefore $xl = 1.2$ in.

$al = \dfrac{874.2}{79.5} + 6 + 1.5 - 1.2 = 17.28$

$a = \dfrac{17.28}{18} = 0.96$

From Table XXII, $C = 0.794$

70XX weld req'd $= \dfrac{79.5}{0.794 \times 18} = 5.57$ sixteenths

Use 3/16 in. fillets all around.

7 **Use 3/8 in. flange plates with 3/16 in. fillet welds to the column.**

LRFD DESIGN EXAMPLE 8

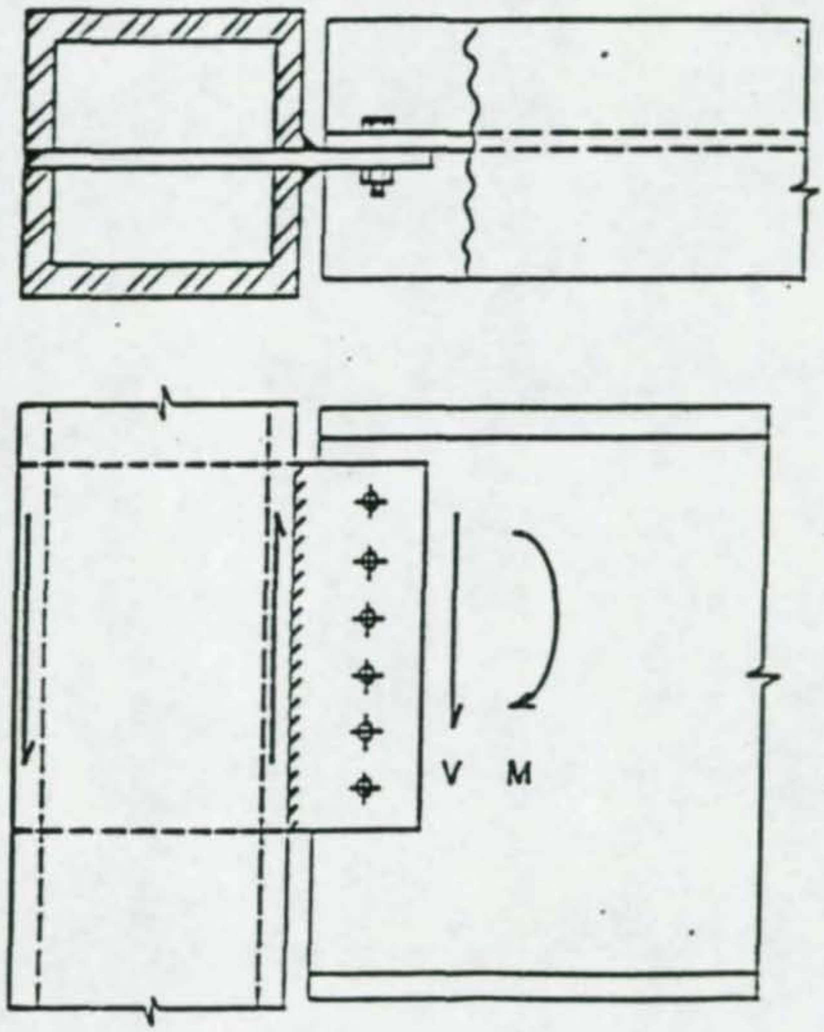
Beam: W24 x 68, A36 Steel

Span: 24 ft., Laterally Supported

Loading: Uniform load with W = 159 K

Column: Tube 12 x 12 x 1/2 in. (F_y = 46 Ksi)

Design the plate welds for a design extending the plate through the column.



Simplified Design Procedure

Step

Design Step 1 through 3 are same as for Example 1, LRFD

The shear and moment at bolt line are

$R = 79.5$ K $M_{bolt} = 715$ K-in.

4 $M_{conn} = 715 + (79.5 \times 3) = 953.5$ K-in.

5 $f_b = 4 \times \dfrac{953.5}{0.375 \times 18^2} = 31.4 < 32.4$ ksi

$f_v = \dfrac{79.4}{0.375 \times 18} = 11.76 < 19.4$ ksi

6 Use LRFD AISC Table XXIII.

$l = 18$ in. and $Kl = 12$ in.

$al = \left(\dfrac{953.5}{79.5}\right) + 6 = 18$ in.

$a = 18.\dfrac{}{18} = 1$, and $K = \dfrac{12}{18} = 0.667$

From Table XXIII, C = 1.019

70XX weld req'd = $\dfrac{79.5}{1.019 \times 18} = 4.35$ sixteenths

Use 5/16 in. fillets on both side of column;

3/8 in. bevel on the opposite side.

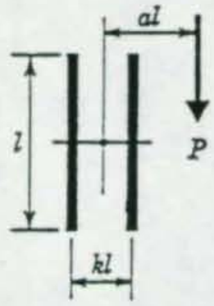
Alternatively, extend plate and use fillet welds on both sides of the column.

APPENDIX B

ASD DESIGN OF WELD GROUPS

ECCENTRIC LOADS ON WELD GROUPS

TABLE XIX Coefficients C



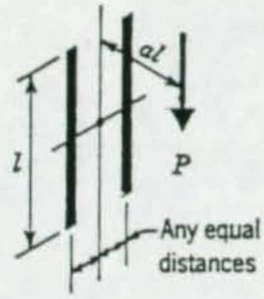
Required Minimum $C = \frac{P}{C_1 D l}$

" " $D = \frac{P}{C C_1 l}$

" " $l = \frac{P}{C C_1 D}$

P = Permissible eccentric load in kips.
l = Length of each weld in inches.
D = Number of sixteenths of an inch in fillet weld size.
C = Coefficients tabulated below.
C_1 = Coefficient for electrode used (see Table on page 4-74).
= 1.0 for E70XX electrodes.

$$P = CC_1Dl$$



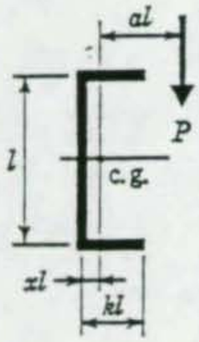
SPECIAL CASE*
(Load not in plane of weld group.)
Use C-values given in column headed k = 0.

a	k															
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
.06	1.67	1.67	1.68	1.68	1.68	1.69	1.69	1.69	1.69	1.70	1.70	1.70	1.71	1.71	1.71	1.71
.08	1.64	1.65	1.65	1.65	1.66	1.66	1.66	1.66	1.67	1.67	1.67	1.67	1.68	1.68	1.69	1.69
.10	1.61	1.61	1.62	1.62	1.62	1.63	1.63	1.63	1.63	1.64	1.64	1.65	1.65	1.66	1.66	1.67
.15	1.51	1.51	1.52	1.52	1.53	1.53	1.54	1.54	1.55	1.56	1.56	1.57	1.58	1.59	1.60	1.61
.20	1.39	1.39	1.40	1.41	1.42	1.43	1.44	1.45	1.46	1.47	1.48	1.50	1.52	1.53	1.54	1.56
.25	1.26	1.27	1.28	1.30	1.31	1.33	1.35	1.36	1.38	1.39	1.41	1.43	1.45	1.47	1.49	1.50
.30	1.14	1.15	1.17	1.19	1.21	1.24	1.26	1.28	1.30	1.32	1.33	1.36	1.39	1.41	1.43	1.45
.40	.939	.951	.976	1.01	1.04	1.07	1.10	1.13	1.16	1.18	1.20	1.24	1.28	1.31	1.33	1.36
.50	.787	.792	.813	.865	.903	.941	.976	1.01	1.04	1.07	1.09	1.14	1.18	1.21	1.25	1.27
.60	.673	.679	.701	.734	.795	.834	.872	.907	.940	.970	.998	1.05	1.09	1.13	1.17	1.20
.70	.585	.592	.615	.647	.708	.748	.787	.823	.857	.888	.918	.971	1.02	1.06	1.10	1.13
.80	.517	.524	.546	.579	.636	.676	.714	.751	.786	.818	.848	.903	.952	.995	1.03	1.07
.90	.463	.469	.491	.524	.576	.615	.654	.690	.725	.757	.788	.844	.893	.938	.978	1.02
1.00	.419	.425	.446	.478	.527	.565	.602	.638	.672	.704	.735	.791	.842	.887	.928	.965
1.20	.351	.357	.377	.406	.448	.484	.519	.553	.586	.617	.647	.702	.752	.798	.840	.878
1.40	.302	.307	.326	.352	.390	.423	.455	.488	.519	.548	.577	.631	.680	.725	.766	.805
1.60	.265	.270	.287	.311	.344	.375	.405	.435	.465	.493	.520	.572	.619	.664	.704	.743
1.80	.236	.241	.256	.278	.308	.336	.365	.393	.421	.448	.474	.523	.569	.612	.652	.689
2.00	.213	.217	.231	.251	.279	.305	.331	.358	.384	.410	.434	.481	.526	.567	.606	.642
2.20	.193	.198	.211	.229	.254	.279	.303	.328	.353	.377	.401	.446	.488	.528	.566	.602
2.40	.177	.181	.194	.211	.234	.256	.280	.303	.327	.350	.372	.415	.456	.495	.531	.566
2.60	.164	.168	.179	.195	.216	.237	.259	.282	.304	.326	.347	.388	.428	.465	.500	.534
2.80	.152	.156	.166	.181	.201	.221	.242	.263	.284	.305	.325	.365	.402	.438	.472	.505
3.00	.142	.145	.155	.169	.188	.207	.226	.246	.266	.286	.306	.344	.380	.415	.448	.479

* Valid only when the connection material between the welds is solid and does not bend in the plane of the welds.

ECCENTRIC LOADS ON WELD GROUPS

TABLE XXIII Coefficients C



P = Permissible eccentric load in kips.
l = Length of weld parallel to load P in inches.
D = Number of sixteenths of an inch in fillet weld size.
C = Coefficients tabulated below.
C_1 = Coefficient for electrode used (see Table on page 4-74).
= 1.0 for E70XX electrodes.
xl = Distance from vertical weld to center of gravity of weld group.

$$P = CC_1Dl$$

Required Minimum $C = \dfrac{P}{C_1Dl}$

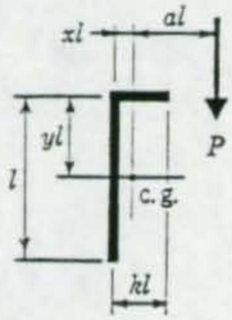
" " $D = \dfrac{P}{CC_1l}$

" " $l = \dfrac{P}{CC_1D}$

a	k															
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
.06	.835	.883	1.05	1.22	1.40	1.58	1.76	1.94	2.12	2.30	2.48	2.84	3.21	3.58	3.95	4.32
.08	.820	.895	1.06	1.23	1.41	1.58	1.76	1.94	2.12	2.30	2.48	2.85	3.21	3.58	3.94	4.31
.10	.804	.902	1.07	1.24	1.41	1.59	1.76	1.94	2.12	2.30	2.48	2.83	3.20	3.56	3.92	4.28
.15	.753	.895	1.06	1.22	1.39	1.56	1.73	1.90	2.07	2.24	2.42	2.76	3.11	3.46	3.81	4.16
.20	.693	.865	1.02	1.18	1.34	1.50	1.67	1.83	1.99	2.16	2.32	2.65	2.99	3.32	3.66	4.00
.25	.630	.823	.972	1.12	1.28	1.43	1.59	1.74	1.90	2.06	2.21	2.53	2.85	3.18	3.51	3.84
.30	.570	.750	.917	1.06	1.21	1.35	1.50	1.65	1.80	1.95	2.10	2.41	2.72	3.04	3.36	3.68
.40	.469	.627	.803	.934	1.07	1.20	1.33	1.47	1.61	1.74	1.89	2.17	2.47	2.77	3.08	3.39
.50	.393	.529	.666	.819	.937	1.06	1.18	1.30	1.43	1.56	1.69	1.96	2.24	2.53	2.83	3.13
.60	.336	.453	.574	.721	.829	.939	1.05	1.17	1.28	1.40	1.53	1.78	2.05	2.32	2.61	2.90
.70	.293	.395	.502	.611	.739	.839	.942	1.05	1.16	1.27	1.39	1.63	1.88	2.14	2.41	2.69
.80	.259	.349	.444	.543	.664	.756	.852	.950	1.05	1.16	1.27	1.49	1.73	1.98	2.24	2.51
.90	.232	.312	.398	.488	.602	.687	.775	.867	.962	1.06	1.16	1.38	1.60	1.84	2.09	2.36
1.00	.209	.282	.360	.442	.550	.629	.711	.796	.885	.978	1.07	1.28	1.49	1.72	1.96	2.21
1.20	.176	.236	.302	.372	.445	.536	.608	.683	.762	.844	.929	1.11	1.31	1.52	1.74	1.97
1.40	.151	.203	.260	.320	.384	.466	.530	.597	.667	.741	.818	.985	1.17	1.36	1.56	1.78
1.60	.132	.178	.228	.281	.338	.412	.469	.529	.593	.660	.731	.883	1.05	1.22	1.41	1.61
1.80	.118	.158	.203	.250	.301	.369	.420	.475	.533	.595	.660	.799	.951	1.11	1.29	1.47
2.00	.106	.142	.182	.225	.272	.334	.381	.431	.484	.541	.601	.730	.870	1.02	1.18	1.35
2.20	.097	.129	.166	.205	.247	.305	.348	.394	.444	.496	.552	.671	.802	.942	1.09	1.25
2.40	.089	.119	.152	.188	.227	.280	.320	.363	.409	.458	.510	.621	.743	.874	1.01	1.16
2.60	.082	.110	.140	.174	.210	.259	.297	.337	.380	.425	.474	.578	.692	.815	.946	1.09
2.80	.076	.102	.130	.161	.195	.242	.277	.314	.354	.397	.442	.540	.647	.763	.886	1.02
3.00	.071	.095	.122	.151	.182	.226	.259	.294	.332	.372	.415	.507	.608	.717	.834	.958
x	0	.008	.028	.056	.088	.125	.163	.204	.246	.289	.333	.423	.515	.609	.704	.800

ECCENTRIC LOADS ON WELD GROUPS

TABLE XXV Coefficients C



P = Permissible eccentric load in kips.
l = Length of weld parallel to load P in inches.
D = Number of sixteenths of an inch in fillet weld size.
C = Coefficients tabulated below.
C_1 = Coefficient for electrode used (see Table on page 4-74).
= 1.0 for E70XX electrodes.
xl = Distance from vertical weld to center of gravity of weld group.
yl = Distance from horizontal weld to center of gravity of weld group.

$$P = CC_1Dl$$

$$\text{Required Minimum } C = \frac{P}{C_1Dl}$$

$$\text{" \quad " } D = \frac{P}{CC_1l}$$

$$\text{" \quad " } l = \frac{P}{CC_1D}$$

a	k															
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
.06	.835	.801	.882	.965	1.05	1.14	1.22	1.31	1.40	1.48	1.57	1.75	1.93	2.11	2.29	2.47
.08	.820	.814	.892	.974	1.06	1.14	1.23	1.32	1.40	1.49	1.58	1.74	1.92	2.10	2.28	2.47
.10	.804	.818	.895	.976	1.06	1.14	1.23	1.31	1.40	1.48	1.57	1.73	1.91	2.10	2.28	2.47
.15	.753	.810	.882	.957	1.03	1.11	1.19	1.27	1.34	1.42	1.51	1.67	1.84	2.09	2.28	2.46
.20	.693	.780	.847	.915	.985	1.06	1.13	1.20	1.27	1.35	1.42	1.58	1.74	1.91	2.28	2.46
.25	.630	.714	.795	.862	.926	.990	1.06	1.12	1.19	1.26	1.34	1.49	1.64	1.80	1.97	2.46
.30	.570	.649	.724	.798	.864	.923	.984	1.05	1.11	1.18	1.25	1.39	1.55	1.70	1.87	2.04
.40	.469	.538	.602	.665	.729	.797	.851	.908	.967	1.03	1.09	1.23	1.38	1.53	1.69	1.85
.50	.393	.452	.507	.562	.618	.676	.740	.791	.846	.904	.964	1.09	1.23	1.38	1.53	1.69
.60	.336	.387	.435	.483	.532	.584	.640	.697	.747	.801	.858	.981	1.11	1.25	1.40	1.55
.70	.293	.337	.379	.421	.465	.512	.563	.620	.667	.718	.771	.887	1.01	1.15	1.29	1.43
.80	.259	.297	.335	.373	.413	.455	.501	.558	.602	.649	.699	.808	.926	1.05	1.19	1.33
.90	.232	.266	.300	.334	.370	.409	.452	.498	.547	.591	.638	.741	.853	.975	1.10	1.24
1.00	.209	.240	.271	.303	.335	.371	.410	.453	.501	.542	.586	.683	.790	.906	1.03	1.16
1.20	.176	.201	.227	.254	.282	.313	.347	.384	.428	.464	.504	.591	.688	.793	.905	1.02
1.40	.151	.173	.196	.219	.243	.270	.300	.333	.373	.406	.441	.520	.607	.703	.806	.915
1.60	.132	.152	.172	.192	.213	.237	.264	.294	.330	.360	.392	.464	.543	.630	.725	.825
1.80	.118	.135	.153	.171	.190	.212	.236	.263	.296	.324	.353	.418	.491	.571	.658	.751
2.00	.106	.122	.138	.154	.172	.191	.213	.238	.269	.294	.321	.380	.447	.521	.602	.688
2.20	.097	.111	.125	.140	.156	.174	.194	.217	.242	.269	.294	.349	.411	.480	.554	.635
2.40	.089	.101	.115	.128	.143	.160	.179	.200	.223	.248	.271	.322	.380	.444	.513	.589
2.60	.082	.094	.106	.119	.132	.148	.165	.185	.207	.230	.251	.299	.353	.413	.478	.548
2.80	.076	.087	.098	.110	.123	.137	.154	.172	.192	.214	.234	.279	.330	.386	.447	.513
3.00	.071	.081	.092	.103	.115	.128	.144	.161	.180	.200	.219	.262	.309	.362	.420	.482
x	0	.004	.016	.034	.057	.083	.112	.144	.177	.213	.250	.327	.408	.492	.578	.666
y	.500	.454	.416	.384	.357	.333	.312	.294	.277	.263	.250	.227	.208	.192	.178	.166

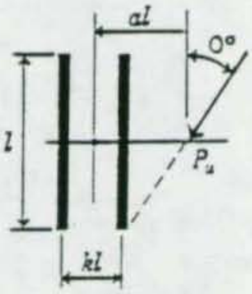
APPENDIX C

LRFD DESIGN OF WELD GROUPS

ECCENTRIC LOADS ON WELD GROUPS

TABLE XVIII Coefficients C

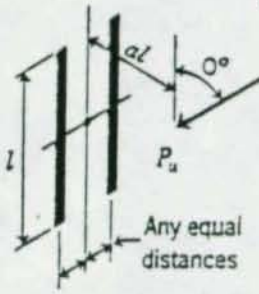
ANGLE = 0°



Required Minimum $C = \dfrac{P_u}{C_1 D l}$

" " $D = \dfrac{P_u}{C C_1 l}$

" " $l = \dfrac{P_u}{C C_1 D}$



P_u = factored eccentric load in kips
l = length of each weld, in.
D = number of sixteenths of an inch in fillet weld size
C = coefficients tabulated below (includes $\phi = 0.75$)
C_1 = coefficient for electrode used (see Table on page 5-89) = 1.0 for E70XX electrodes

$P_u = C C_1 D l$

SPECIAL CASE*
(Load not in plane of weld group)
Use C-values given in column headed $k = 0$

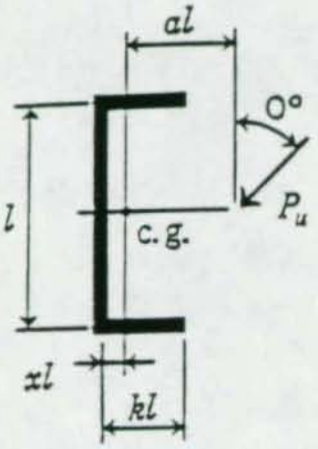
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
.05	2.743	2.743	2.744	2.745	2.747	2.751	2.754	2.757	2.761	2.763	2.766	2.770	2.773	2.775	2.777	2.778
.10	2.627	2.628	2.631	2.636	2.645	2.656	2.668	2.681	2.692	2.703	2.713	2.728	2.740	2.748	2.755	2.760
.15	2.459	2.461	2.468	2.480	2.497	2.517	2.540	2.564	2.587	2.608	2.628	2.661	2.685	2.704	2.718	2.729
.20	2.264	2.268	2.281	2.300	2.325	2.354	2.387	2.420	2.455	2.487	2.518	2.571	2.612	2.644	2.668	2.687
.25	2.064	2.071	2.089	2.117	2.150	2.186	2.226	2.267	2.309	2.351	2.391	2.464	2.523	2.570	2.606	2.635
.30	1.874	1.883	1.907	1.942	1.982	2.026	2.070	2.116	2.162	2.209	2.213	2.344	2.421	2.483	2.533	2.572
.40	1.549	1.561	1.593	1.638	1.691	1.746	1.799	1.850	1.900	1.941	1.987	2.080	2.137	2.157	2.202	2.422
.50	1.299	1.312	1.348	1.400	1.460	1.522	1.581	1.638	1.691	1.741	1.786	1.871	1.956	2.005	2.049	2.111
.60	1.107	1.123	1.160	1.216	1.279	1.345	1.408	1.468	1.525	1.577	1.625	1.713	1.790	1.865	1.933	1.969
.70	.962	.975	1.015	1.072	1.138	1.204	1.268	1.329	1.387	1.442	1.493	1.583	1.663	1.733	1.797	1.858
.80	.849	.862	.902	.960	1.025	1.090	1.154	1.215	1.272	1.326	1.377	1.475	1.554	1.626	1.691	1.748
.90	.758	.771	.811	.868	.932	.996	1.060	1.120	1.177	1.229	1.279	1.373	1.464	1.533	1.601	1.658
1.00	.684	.698	.737	.791	.853	.916	.979	1.039	1.095	1.148	1.197	1.287	1.371	1.458	1.517	1.580
1.20	.572	.586	.622	.672	.729	.789	.847	.905	.961	1.014	1.062	1.149	1.227	1.301	1.369	1.454
1.40	.491	.504	.538	.583	.635	.690	.745	.800	.854	.905	.954	1.038	1.114	1.185	1.250	1.312
1.60	.430	.443	.474	.514	.561	.612	.665	.715	.767	.817	.864	.947	1.023	1.090	1.153	1.211
1.80	.383	.395	.423	.460	.503	.550	.598	646	.695	.742	.788	.870	.945	1.012	1.071	1.127
2.00	.345	.356	.382	.416	.455	.498	.544	.590	.634	.680	.724	.804	.878	.944	1.003	1.057
2.20	.314	.324	.348	.379	.415	.455	.498	.541	.583	.626	.668	.749	.819	.885	.944	.997
2.40	.287	.298	.319	.348	.382	.419	.459	.499	.541	.580	.620	.698	.767	.832	.891	.943
2.60	.265	.275	.295	.322	.353	.388	.425	.463	.502	.540	.578	.653	.720	.784	.843	.896
2.80	.246	.255	.274	.299	.329	.361	.396	.432	.469	.506	.541	.613	.681	.741	.799	.852
3.00	.230	.239	.256	.280	.307	.338	.371	.405	.439	.475	.508	.577	.643	.702	.759	.812

*Valid only when the connection material between the welds is solid and does not bend in the plane of the welds.

ECCENTRIC LOADS ON WELD GROUPS

TABLE XXII Coefficients C

ANGLE = 0°



P_u = factored eccentric load in kips
l = length of each weld, in.
D = number of sixteenths of an inch in fillet weld size
C = coefficients tabulated below (includes $\phi = 0.75$)
C_1 = coefficient for electrode used (see Table on page 5-89) = 1.0 for E70XX electrodes
xl = distance from vertical weld to center of gravity of weld group

$$P_u = CC_1Dl$$

Required Minimum $C = \dfrac{P_u}{C_1Dl}$

" " $D = \dfrac{P_u}{CC_1l}$

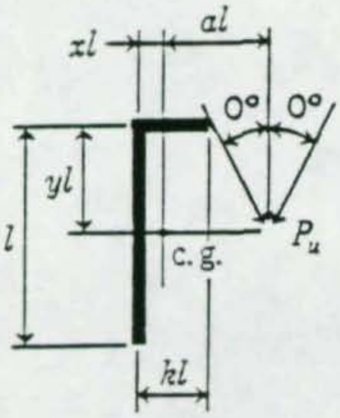
" " $l = \dfrac{P_u}{CC_1D}$

a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
.05	1.371	1.652	1.930	2.208	2.486	2.764	3.041	3.319	3.597	3.875	4.153	4.709	5.266	5.823	6.380	6.938
.10	1.314	1.599	1.877	2.153	2.429	2.704	2.979	3.255	3.531	3.808	4.084	4.640	5.196	5.754	6.312	6.871
.15	1.229	1.518	1.795	2.068	2.339	2.611	2.882	3.154	3.427	3.701	3.976	4.529	5.084	5.642	6.202	6.762
.20	1.132	1.421	1.694	1.962	2.228	2.494	2.760	3.027	3.295	3.565	3.836	4.383	4.936	5.493	6.053	6.616
.25	1.032	1.318	1.585	1.845	2.104	2.362	2.621	2.881	3.143	3.408	3.674	4.213	4.760	5.314	5.873	6.436
.30	.937	1.215	1.473	1.725	1.975	2.225	2.475	2.728	2.983	3.240	3.500	4.027	4.566	5.113	5.669	6.231
.40	.774	1.028	1.265	1.497	1.726	1.957	2.190	2.425	2.663	2.904	3.148	3.647	4.159	4.685	5.223	5.773
.50	.649	.876	1.090	1.300	1.509	1.720	1.935	2.152	2.373	2.598	2.826	3.293	3.774	4.270	4.760	5.278
.60	.553	.756	.948	1.137	1.328	1.521	1.718	1.920	2.125	2.334	2.547	2.985	3.437	3.903	4.371	4.858
.70	.481	.662	.834	1.006	1.179	1.357	1.538	1.724	1.915	2.110	2.310	2.723	3.152	3.596	4.053	4.521
.80	.424	.585	.742	.899	1.057	1.221	1.388	1.560	1.738	1.922	2.110	2.503	2.913	3.339	3.780	4.232
.90	.379	.525	.668	.811	.957	1.107	1.262	1.423	1.590	1.763	1.941	2.314	2.709	3.121	3.546	3.983
1.00	.342	.475	.606	.737	.872	1.012	1.156	1.307	1.464	1.627	1.796	2.154	2.533	2.930	3.342	3.768
1.20	.286	.398	.510	.623	.740	.861	.989	1.122	1.263	1.410	1.563	1.890	2.238	2.605	2.988	3.386
1.40	.246	.343	.440	.539	.642	.749	.863	.983	1.110	1.243	1.384	1.682	2.003	2.342	2.697	3.068
1.60	.215	.301	.386	.474	.566	.663	.765	.874	.990	1.112	1.240	1.514	1.808	2.121	2.450	2.795
1.80	.191	.268	.344	.423	.506	.594	.687	.787	.893	1.005	1.123	1.374	1.645	1.935	2.241	2.562
2.00	.172	.241	.311	.382	.458	.538	.624	.715	.813	.916	1.025	1.257	1.508	1.776	2.061	2.361
2.20	.157	.219	.283	.349	.418	.492	.571	.656	.746	.841	.942	1.157	1.390	1.640	1.906	2.186
2.40	.144	.201	.260	.320	.384	.453	.526	.605	.689	.778	.871	1.072	1.289	1.522	1.771	2.039
2.60	.133	.187	.240	.296	.356	.420	.488	.562	.640	.723	.810	.997	1.201	1.419	1.657	1.904
2.80	.123	.173	.223	.275	.331	.391	.455	.524	.597	.675	.757	.932	1.123	1.332	1.552	1.786
3.00	.115	.161	.208	.257	.310	.366	.426	.491	.560	.633	.710	.875	1.057	1.252	1.459	1.680
x	.000	.008	.029	.056	.089	.125	.164	.204	.246	.289	.333	.424	.516	.610	.704	.800

ECCENTRIC LOADS ON WELD GROUPS

TABLE XXIV Coefficients C

ANGLE = 0°



P_u = factored eccentric load in kips
l = length of each weld, in.
D = number of sixteenths of an inch in fillet weld size
C^* = coefficients tabulated below (includes $\phi = 0.75$)
C_1 = coefficient for electrode used (see Table on page 5-89) = 1.0 for E70XX electrodes
xl = distance from vertical weld to center of gravity of weld group
yl = distance from horizontal weld to center of gravity of weld group

$$P_u = CC_1Dl$$

Required Minimum $C = \dfrac{P_u}{C_1Dl}$

" " $D = \dfrac{P_u}{CC_1l}$

" " $l = \dfrac{P_u}{CC_1D}$

a	k															
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
.05	1.371	1.512	1.651	1.789	1.927	2.065	2.204	2.342	2.481	2.620	2.760	3.039	3.318	3.598	3.878	4.158
.10	1.314	1.456	1.594	1.730	1.866	2.001	2.137	2.274	2.411	2.550	2.689	2.970	3.253	3.537	3.821	4.105
.15	1.229	1.373	1.509	1.642	1.774	1.906	2.038	2.171	2.307	2.444	2.582	2.864	3.151	3.439	3.729	4.019
.20	1.132	1.276	1.408	1.537	1.663	1.790	1.918	2.047	2.179	2.313	2.450	2.731	3.019	3.312	3.608	3.904
.25	1.032	1.173	1.301	1.424	1.545	1.666	1.788	1.913	2.041	2.171	2.305	2.582	2.869	3.164	3.463	3.765
.30	.937	1.074	1.196	1.313	1.427	1.542	1.659	1.779	1.901	2.027	2.157	2.427	2.710	3.002	3.303	3.608
.40	.774	.898	1.008	1.111	1.213	1.316	1.422	1.531	1.643	1.757	1.871	2.119	2.379	2.654	2.941	3.188
.50	.649	.759	.856	.948	1.038	1.130	1.225	1.324	1.426	1.531	1.637	1.864	2.104	2.356	2.620	2.885
.60	.553	.652	.738	.819	.900	.982	1.068	1.158	1.251	1.348	1.449	1.664	1.888	2.125	2.370	2.625
.70	.481	.567	.643	.717	.790	.864	.942	1.024	1.110	1.202	1.297	1.500	1.716	1.942	2.177	2.420
.80	.424	.501	.570	.636	.702	.769	.841	.917	.998	1.083	1.173	1.366	1.573	1.792	2.019	2.254
.90	.379	.448	.511	.569	.630	.692	.758	.830	.906	.986	1.071	1.255	1.452	1.664	1.883	2.110
1.00	.342	.405	.462	.516	.571	.629	.691	.758	.829	.905	.986	1.160	1.351	1.553	1.764	1.982
1.20	.286	.339	.387	.434	.481	.531	.586	.644	.709	.777	.850	1.010	1.182	1.366	1.560	1.762
1.40	.246	.292	.333	.374	.415	.460	.508	.561	.619	.681	.747	.891	1.049	1.219	1.398	1.586
1.60	.215	.256	.293	.329	.365	.405	.449	.497	.549	.605	.665	.798	.942	1.098	1.263	1.436
1.80	.191	.227	.261	.293	.326	.362	.402	.445	.493	.545	.600	.721	.853	.996	1.150	1.312
2.00	.172	.205	.235	.264	.294	.327	.364	.404	.448	.495	.545	.656	.778	.912	1.054	1.204
2.20	.157	.188	.214	.241	.268	.299	.332	.369	.410	.453	.500	.602	.716	.840	.972	1.112
2.40	.144	.171	.196	.221	.246	.275	.305	.340	.377	.418	.461	.557	.662	.777	.900	1.033
2.60	.133	.158	.181	.204	.228	.254	.283	.315	.350	.388	.428	.517	.615	.723	.839	.963
2.80	.123	.146	.168	.190	.212	.237	.263	.293	.326	.362	.399	.483	.574	.676	.785	.902
3.00	.115	.137	.157	.177	.198	.221	.246	.275	.306	.339	.374	.453	.538	.635	.737	.848
x	.000	.005	.017	.035	.057	.083	.113	.144	.178	.213	.250	.327	.408	.492	.579	.667
y	.500	.455	.417	.385	.357	.333	.313	.294	.278	.263	.250	.227	.208	.192	.179	.167

*Tabulated values are the minimum of either − or + load angles.