CONNECTION LIMIT STATES DESIGN TEACHING AID

by

Robert A. Kerr

Project and report submitted to the Faculty of the
Virginia Polytechnic Institute and State University
in partial fulfillment of the requirements for the degree of
MASTER OF ENGINEERING
in
CIVIL ENGINEERING

APPROVED:

W. Samuel Easterling

W. Samuel Easterling, Chairman

Thomas M. Murray

Thomas M. Murray

Don A. Garst

Don A. Garst

July, 1994

Blacksburg, Virginia

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W. Samuel Easterling, Chairman

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(ABSTRACT)

Steel connection design is one area in structural steel design courses that is not always thoroughly addressed. This report attempts to address this area of steel design at a basic level. Its purpose is to be used as a teaching aid for a structural steel design course, and to familiarize students with connection design and its associated strength limit states.

Limit states for steel connection design have been covered using both AISC ASD and LRFD Specifications. However, all included connection design examples used only LRFD limit states. Wherever possible all limit state calculations are accompanied by print-outs from a knowledge-based expert system, CONXPRT.

Typical building connection limit states are covered by way of an accompanying steel structure, which includes many of the connections, in order for students to receive an adequate grasp of both simple framing and moment connections. The purpose of this steel "sculpture" is to actively reinforce the students' understanding of the basic building connections seen in industry today.

ACKNOWLEDGMENTS

The author wishes to express his gratitude to his major advisor, Dr. W. S. Easterling, and to the other members of his committee for their aid and guidance in the preparation of this report. The author is also grateful to his entire family for their encouragement, continued love, and support. The author would also like to express his gratitude to all of his friends for their invaluable assistance and friendship. Finally, the author would like to dedicate this project in memory of his father, Jon A. Kerr.

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CHAPTER I
INTRODUCTION

Steel connection design is an important aspect in structural steel design. However, many times design engineers do not address connection design directly, leaving the design responsibility to the steel fabricator. Similarly, steel connection design is one area in academic structural steel design courses that is not thoroughly addressed. This practice possibly creates a problem in industry. By allowing structural engineers to ignore connection design, potential strength or safety concerns could be overlooked . This report attempts to address this problem at a basic steel design level. Its purpose is to be used as a teaching aid for a basic level steel design course, familiarizing students with steel connection design and the associated strength limit states.

Over twenty building connections will be covered in this report to give students an adequate grasp of both simple framing and moment connections used in industry. Wherever possible, the design aid will include computer print-outs from a knowledge-based expert connection design system, CONXPRT, to accompany manually calculated design limit states (CONXPRT 1992). Basic connection design limit state criteria will be covered in both LRFD and ASD (AISC 1989, 1993). However, all connections will be designed using LRFD (AISC 1986, 1993).

To present connection design in a practical way, the building connections will come directly from a designed structural steel "sculpture". This "sculpture" is accompanied by fabrication and erection drawings so that it may be constructed and viewed by students. The purpose of this "sculpture" is to actively reinforce the students' understanding of the basic building connections seen in industry today.

CHAPTER II
CONNECTION DESIGN BASICS

2.1 Connection Classification

Connection behavior is often defined by its moment-rotation relationship or M-ϕ diagram, as seen in Fig. 2.1. The slope of the M-ϕ curve is an indication of the rotational stiffness of the connection. The greater the slope of the curve, the greater the stiffness of the connection.

This stiffness is expressed by three types of construction as defined by the AISC ASD Specification (AISC 1989): Type I, Type II, and Type III. Type I Construction (Rigid framing) assumes that the connections have sufficient rigidity to fully restrain rotation at joints. Type II Construction (Simple framing) assumes that the connections are "pinned" or free to rotate, and only transfer shear. Type III Construction (Semi-rigid framing) is somewhere in between Type I and Type II.

The AISC LRFD Specification (AISC 1993) defines two types of construction: FR and PR. FR construction (Fully restrained) is the same as ASD Type I. PR construction (Partially restrained) contains both ASD Type II and III.

2.2 Basic Design Criteria

The basic design criteria in LRFD connection design is that the factored load, R_u (according to LRFD-A4.1) acting upon a connection may not exceed the connection design strength, ϕR_n.

$$R_u \leq \phi R_n \tag{2.1}$$

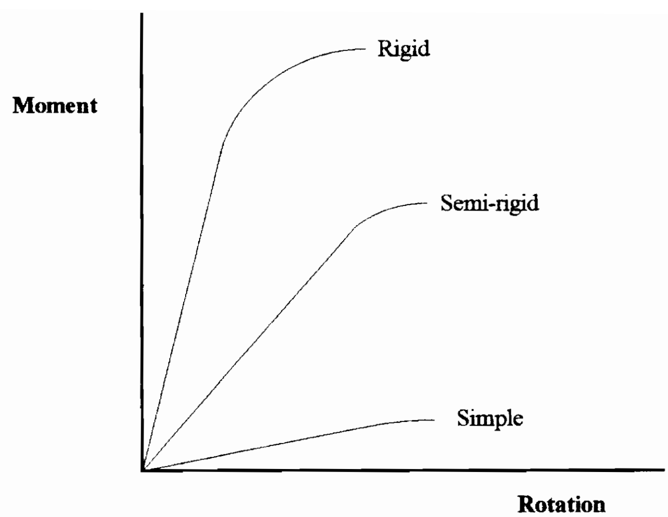


Figure 2.1 Moment-Rotation Relationship for Typical Connections (AISC 1992)

The basic design criteria in ASD connection design is that the nominal stress, f (according to ASD-A4.1) acting upon a connection must be less than or equal to the connection allowable stress, F, which includes a factor of safety.

$$f \leq F \tag{2.2}$$

2.3 Connection Limit States

The following connection limit states will be presented according the latest editions of both the AISC LRFD and ASD Specifications (Murray 1993).

2.3.1 Tension Yield

One of the controlling limit states for tension members is tension yielding of the gross cross-section away from the connection, as in Fig. 2.2. For a tension member without bolt holes, the nominal strength or allowable stress is as follows:

LRFD: $$T_u \leq \phi T_n = \phi F_y A_g \tag{2.3}$$

$$\phi = 0.9$$

ASD: $$f_t = T / A_g \leq F_t = 0.6F_y \tag{2.4}$$

2.3.2 Tension Rupture

Tension members with bolt holes have a reduced cross-sectional area referred to as the *net area*, A_n. Concentrated stresses occur at bolt holes in tension members which cause localized failure. These stresses result in tension rupture through the *effective net area*, A_e, of a tension member (Salmon and Johnson 1990). Therefore, the limit state becomes:

LRFD: $$T_u \leq \phi T_n = \phi F_u A_e \tag{2.5}$$

$$\phi = 0.75$$

ASD: $$f_t = T / A_e \leq F_t = 0.5F_u \tag{2.6}$$

where, $A_e = UA_n$

$A_n = A_g - \sum A_{hole}$

U = reduction factor from ASD-B3 or LRFD-B3.

2.3.3 Shear Yield

For connection members that are subjected to shear, the limit state is shear yielding of the gross cross-section through the member, as shown in Fig. 2.3. Therefore, for connection members without bolt holes, the nominal shear strength or allowable stress is as follows:

LRFD: $$V_u \leq \phi V_n = \phi 0.6 F_y A_g \qquad (2.7)$$

$\phi = 0.9$

ASD: $$f_v = V / A_g \leq F_v = 0.4 F_y \qquad (2.8)$$

2.3.4 Shear Rupture

Connection members with bolt holes have a reduced cross-sectional area referred to as the *net area*, A_n. Again, localized stresses result in shear rupture through the *net area* of the member. Therefore the limit state becomes:

LRFD: $$V_u \leq \phi V_n = \phi 0.6 F_u A_n \qquad (2.9)$$

$\phi = 0.75$

ASD: $$f_v = V / A_n \leq F_v = 0.3 F_u \qquad (2.10)$$

2.3.5 Fillet Weld Rupture

The fillet weld design strength is based on the shear resistance through the throat of the weld. Assuming SMAW, the limit state is as follows:

LRFD: $$R_u \leq \phi R_n = \phi 0.6 F_{uexx} A_{weld} \qquad (2.11)$$

$\phi = 0.75$

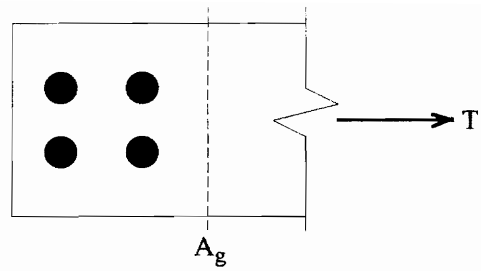


Figure 2.2 Tension Yield Limit State

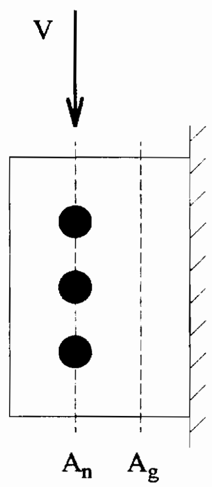


Figure 2.3 Shear Yield Limit State

Assuming 1/16 inch fillet, E70xx electrodes-

$$\phi R_n = 0.75(0.6)(70)(0.707)(1/16)$$

$$= 1.392 \text{ k/in/(1/16)} \tag{2.11a}$$

ASD: $$f_{vw} \leq F_v = 0.3F_{uexx}A_{weld} \tag{2.12}$$

Assuming 1/16 inch fillet, E70xx electrodes-

$$F_v = 0.3(70)(0.707)(1/16)$$

$$= 0.928 \text{ k/in/(1/16)} \tag{2.12a}$$

To obtain the weld rupture strength for a particular fillet weld, Eq. 2.11a and Eq. 2.12a must be multiplied by the weld length, L_{weld}, and the number of 16^{th} inches in the weld size.

It should be noted that the design strength of the fillet weld per unit length must not be less than the shear rupture strength per unit length of the adjacent base metal.

2.3.6 Full and Partial Penetration Groove Welds

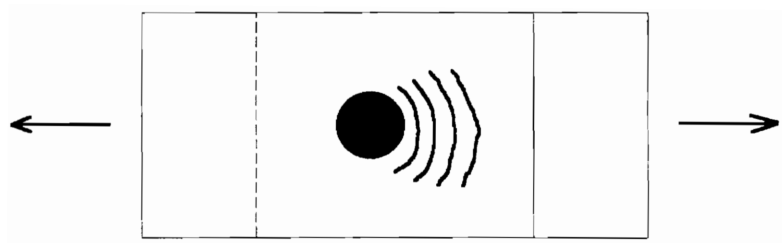
The design criteria for full and partial penetration welds is covered in both the ASD and LRFD Specifications. When using full or partial penetration welds, the designer should use a weld metal that is comparable to that of the base metal.

LRFD: See LRFD Table J2.5

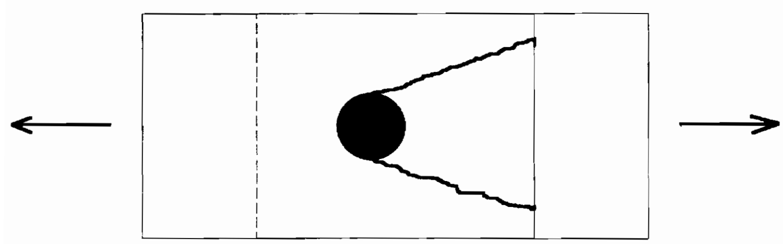
ASD: See ASD Table J2.5

2.3.7 Plate Bearing / Tear-Out

The bearing/tear-out limit state is related to the deformation around a bolt hole. The bearing/tear-out strength is the force applied by the bolt or fastener against the side of the hole which will split or tear the plate as seen in Fig. 2.4. If the distance between the center of the hole to either the edge of the plate or an adjacent hole is large, the less the possibility of having a bearing/tear-out type failure (Salmon and Johnson 1990). The following limit states apply:



(a) Bearing Type Failure



(b) Tear-out Type Failure

Figure 2.4 Plate Bearing / Tear-out Failure Limit State

For edge distance, $e \geq 1.5d_b$ and spacing, $s \geq 3d_b$ (only bearing applies)

LRFD:	$R_u \leq \phi R_n = \phi \sum_{bolt}(2.4F_u d_b t)$	STD, OVS, SS, LS	(2.13)
	$R_u \leq \phi R_n = \phi \sum_{bolt}(2.0F_u d_b t)$	LS Normal	(2.14)
	$\phi = 0.75$		
ASD:	$f_b = V / (d_b t) \leq F_b = \sum_{bolt}(1.2F_u)$	STD, OVS, SS, LS	(2.15)
	$f_b = V / (d_b t) \leq F_b = \sum_{bolt}(1.0F_u)$	LS Normal	(2.16)

For edge distance, $e < 1.5d_b$ or spacing, $s < 3d_b$ (tear-out applies)

LRFD:

Ext. Bolts	$R_n = F_u et \leq 2.4F_u d_b t$	STD, OVS, SS, LS	(2.17)
	$R_n = F_u et \leq 2.0F_u d_b t$	LS Normal	(2.17a)
Int. Bolts	$R_n = F_u(s - d_h / 2)t \leq 2.4F_u d_b t$	STD, OVS, SS, LS	(2.18)
	$R_n = F_u(s - d_h / 2)t \leq 2.0F_u d_b t$	LS Normal	(2.18a)

$$\therefore \quad \phi R_n = \phi[\textstyle\sum_{ext\ bolt}(R_n \text{ ext bolt eq.}) + \sum_{int\ bolt}(R_n \text{ int bolt eq.})] \qquad (2.19)$$

$\phi = 0.75$

ASD:

Ext. Bolts	$F_b = 0.5F_u et \leq 1.2F_u$	STD, OVS, SS, LS	(2.20)
	$F_b = 0.5F_u et \leq 1.0F_u$	LS Normal	(2.21)
Int. Bolts	$F_b = 0.5F_u(s - d / 2)t \leq 1.2F_u$	STD, OVS, SS, LS	(2.22)
	$F_b = 0.5F_u(s - d / 2)t \leq 1.0F_u$	LS Normal	(2.23)

$$\therefore \quad F_b = [\textstyle\sum_{ext\ bolt}(F_b \text{ ext bolt eq.}) + \sum_{int\ bolt}(F_b \text{ int bolt eq.})] \qquad (2.24)$$

2.3.8 Bolt Shear

The bolt shear limit state is related to the rupture and failure mode shown in Fig. 2.5. It is important to note the number of shear planes, *m*, in the connection for this limit state. The limit state is as follows:

LRFD: $$R_u \leq \phi R_b = m\phi r_v A_b \tag{2.25}$$

ϕr_v from LRFD Table J3.2

ASD: $$f_v \leq F_v \tag{2.26}$$

F_v from ASD Table J3.2

2.3.9 Combined Shear and Tension on Bolts

In many connections, both shear and tension occur and must be considered in design. An example of this type of loading is shown by the bearing-type connection in Fig. 2.6, where the bottom fasteners are subjected to both shear and tension. When considering both shear and tension in a bolted connection, there are two major connection types: Bearing and Slip-Critical.

Bearing-type connection limit states are based on an interaction relationship between shear and tension stresses, as seen in Fig. 2.7 for A325-N bolts. Slip-critical connection limit states similarly use a straight-line interaction relationship, but are more conservative than the bearing-type. Slip-critical connections use service loads in their design, because slip resistance is a service load consideration.

The following limit states apply for **bearing-type** connections.

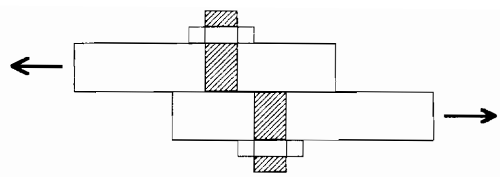
LRFD: See LRFD Spec. J3.7 & Table J3.2

$f_v \leq \phi F_v$ from Table J3.2

$$\phi R_n = \phi F_t A_b, \quad \phi = 0.75 \tag{2.27}$$

ASD: See ASD Spec. J3.5 & Table J3.3

$$f_v \leq F_v \text{ from Table J3.2} \tag{2.28}$$



Shear planes, m = 1

Figure 2.5 Bolt Shear Failure Limit State

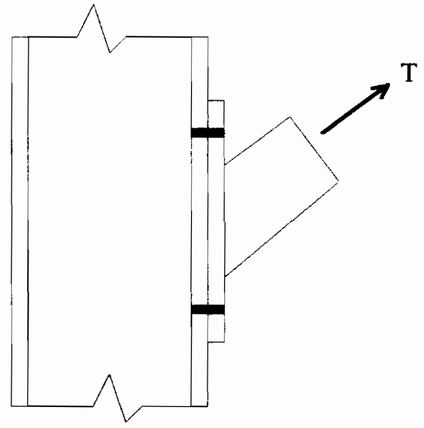


Figure 2.6 Combined Shear and Tension Connection

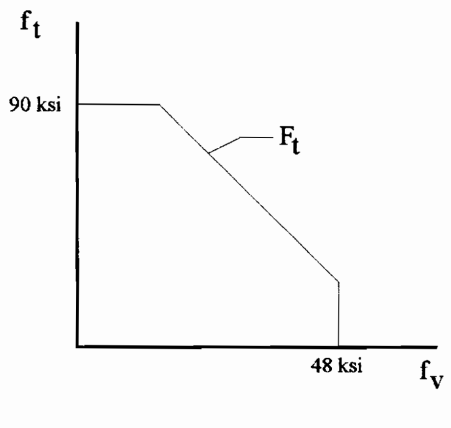


Figure 2.7 Shear and Tension Interaction Curve for A325-N bolts - LRFD

The following limit states apply for **slip-critical** connections.

LRFD:

Service loads
(J3.9a) $\phi r_v = \phi F_v A_b(1 - T / T_b)$ (2.29)

F_v from Table J3.6, T_b from Table J3.1

$\phi = 1.0$ ($\phi = 0.85$ for LS parallel to load)

Factored loads
(J3.9b) $\phi R_n = 1.13\mu T_m N_b N_s(1 - T_u / 1.13 T_m N_b)$ (2.30)

ϕ varies with hole type

ASD: See ASD Spec. J3.6

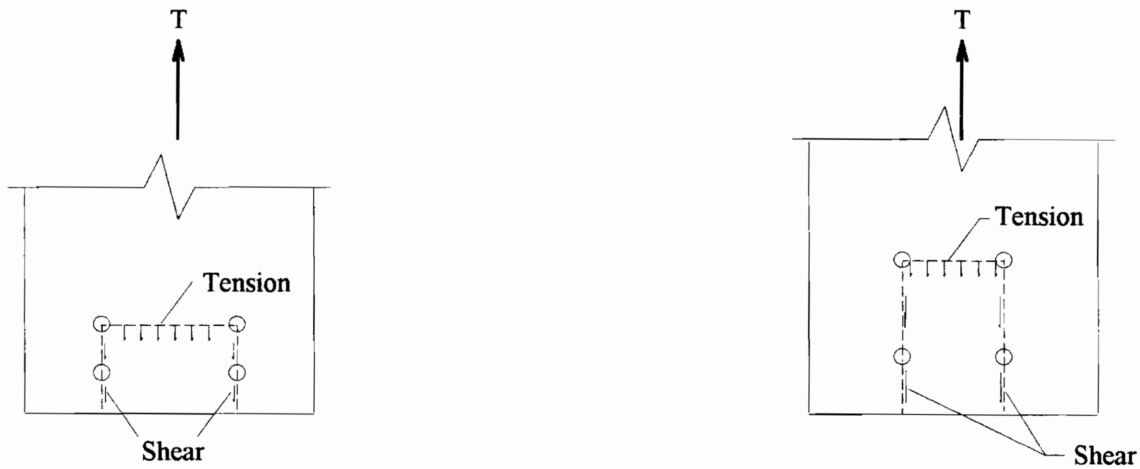
$f_v \leq (1 - f_t A_b / T_b) F_v$ (2.31)

F_v from Table J3.2, T_b from Table J3.7

2.3.10 Block Shear

Block shear is a rupture failure limit state that may control in either bolted or welded connections. Failure occurs with shear along a line of fasteners, combined with tension acting along a line perpendicular to the line of force. Rupture does not occur on both lines of force simultaneously. The failure consists of a rupture on one section with yielding occurring on another. Fig. 2.8 illustrates the block shear limit state in a direct loaded connection. In Fig. 2.8a, the probable failure mode would be tension rupture and shear yield, because the section in tension is larger than the one in shear. The opposite scenario is shown in Fig. 2.8b.

Direct-loaded and coped-beam connections are the two primary connection limit states that will be addressed (Murray 1993).



(a) Large tension, small shear

(b) Large shear, small tension

Figure 2.8 Block Shear Failure Limit State (Salmon and Johnson 1990)

2.3.10.1 Block shear in Direct-Loaded Connections

Fig. 2.9 shows both bolted and welded connections in direct-load. The following limit states apply.

LRFD: See LRFD Spec. J4.3

$$T_n = \left\{ \max \left| \begin{array}{l} \text{shear rupture} \\ \text{tension rupture} \end{array} \right. \right\} + \text{ opposite yield str} \qquad (2.32)$$

shear rupture $= 0.6F_uA_{nv}$ shear yield $= 0.6F_yA_{gv}$

tension rupture $= F_uA_{nt}$ tension yield $= F_yA_{gt}$

$\phi = 0.75$

ASD: See ASD Spec. J4

$$t \leq T_{bs} = 0.5F_uA_{nt} + 0.3F_uA_{nv} \qquad (2.33)$$

2.3.10.2 Block shear in Coped-Beam Connections

Typically, a portion of a framing beam flange and web must be cut in order for it to frame into a supporting girder at the same elevation. This type of construction is typical to bolted, as well as welded beam web connections. In both cases, the block shear limit state must be investigated. The following limit states apply for this connection type.

Welded Connection (Ref. Fig. 2.10)

LRFD: $$\phi R_n = 0.6F_y l t_w + (F_u a t_w / 2) \qquad (2.34)$$

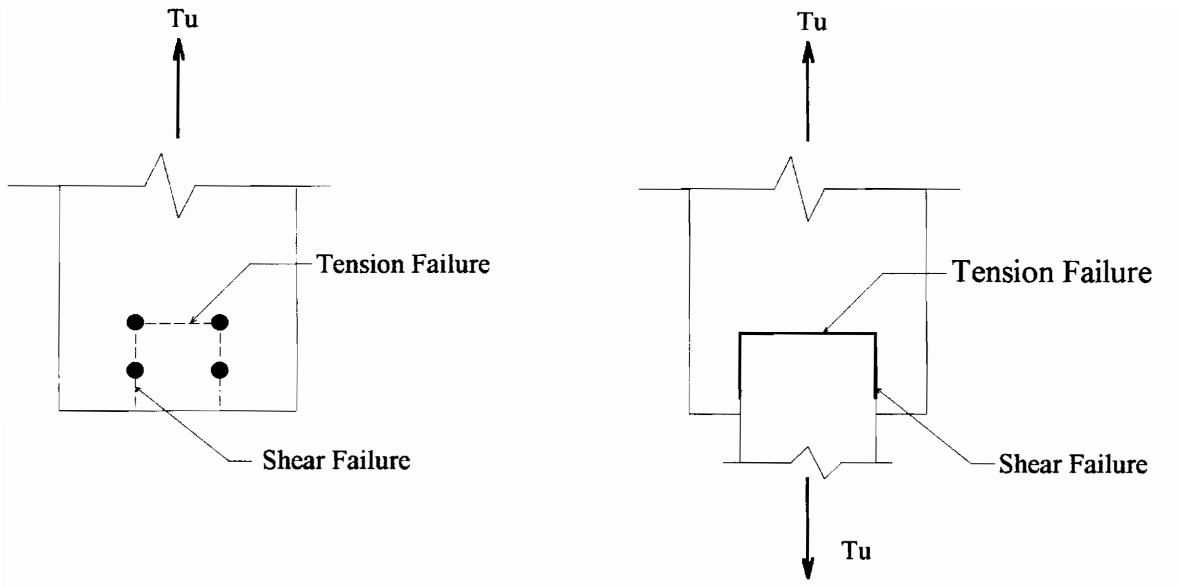
$\phi = 0.75$

ASD: $$R_{bs} = 0.4F_y l t_w + (0.5F_u a t_w / 2) \qquad (2.35)$$

Bolted Connection (Ref. Fig. 2.10)

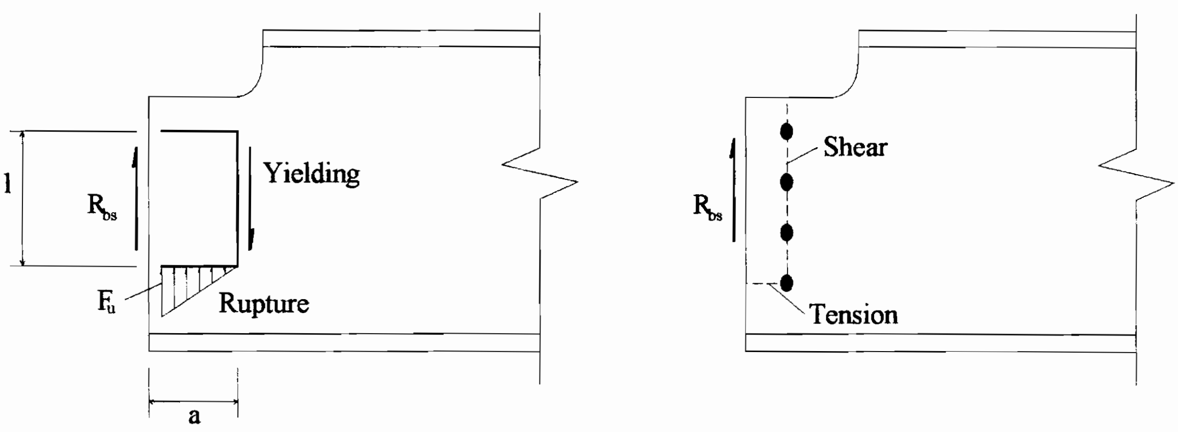
LRFD: See Limit State for Direct-Load Connection

ASD: See Limit State for Direct-Load Connection



(Bolted) (Welded)

Figure 2.9 Block Shear - Direct-Loaded Connection



(Welded) (Bolted)

Figure 2.10 Block Shear - Coped Beam Connection

2.3.11 Coped-Beam Strength

In addition to block shear failure in coped beam connections, either local yielding or buckling of the beam web may occur at a critical section shown in Fig. 2.11. This type of failure can occur in beams with top and bottom copes. For both situations, the limit state check treats the coped beams as tee-sections subjected to a moment at a distance c, from the end of the beam. The following limit states will address top copes, bottom copes, and finally top and bottom copes combined.

The limit states for beams with only a **Bottom Cope** as shown in Fig. 2.3.11a are:

LRFD: $$V_u \leq \phi V_n = (0.9F_y)S_{tee} / e \tag{2.36}$$

ASD: $$f_b = Ve / S_{tee} \leq F_b = 0.6F_y \tag{2.37}$$

The limit states for beams with only a **Top Cope** as shown in Fig. 2.3.11b are:

LRFD: LRFD Volume II App. B

Yielding $$V_u \leq \phi V_n = (0.9F_y)S_{tee} / e \tag{2.38}$$

Buckling $$V_u \leq \phi V_n = (0.9F_{cr})S_{tee} / e \tag{2.39}$$

$$F_{cr} = \frac{\pi^2 E}{12(1-\upsilon^2)}\left(\frac{t_w}{h_o}\right)^2 fk \tag{2.40}$$

where,

$$k = 2.2\left(\frac{h_o}{c}\right)1.65 \quad \text{if } \frac{c}{h_o} \leq 1.0$$

$$k = 2.2\left(\frac{h_o}{c}\right) \quad \text{if } \frac{c}{h_o} \geq 1.0$$

$$f = 2\frac{c}{d} \quad \text{if } \frac{c}{d} \leq 1.0$$

$$f = 1 + \frac{c}{d} \quad \text{if } \frac{c}{d} \geq 1.0$$

Note: When E = 29,000 ksi and $\upsilon = 0.3$,

$$F_{cr} = 26,200\left(\frac{t_w}{h_o}\right)^2 fk \qquad (2.41)$$

ASD: ASD Volume II App. B

Yielding $f_b = Ve / S_{tee} \leq 0.6F_y$ (2.42)

Buckling $f_b = Ve / S_{tee} \leq F_b$ (2.43)

$F_b = F_{cr} / F.S.$, F.S. = 1.67

where, F_{cr} same as for LRFD

Note: When F.S. = 1.67, E = 29,000 ksi and $\upsilon = 0.3$,

$$F_{cr} = 15,700\left(\frac{t_w}{h_o}\right)^2 fk \qquad (2.44)$$

The limit states for beams with only a **Double Cope** as shown in Fig. 2.3.11c are:

LRFD: Ref. Volume II App. B

Yielding $V_u \leq \phi V_n = (0.9F_y)S_{pl} / e$ (2.45)

Buckling $V_u \leq \phi V_n = (0.9F_{cr})S_{pl} / e$ (2.46)

$$F_{cr} = 0.62\pi E \frac{t_w}{ch_o}^2 f_d \qquad (2.47)$$

where, $f_d = 3.5 - 7.5(d_c / d)$

Note: When E = 29,000 ksi,

$$F_{cr} = 56,500 \frac{t_w}{ch_o}^2 (3.5 - 7.5\, d_c / d) \qquad (2.48)$$

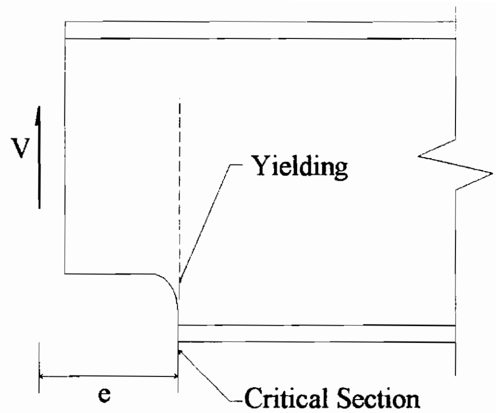
ASD: Ref. Volume II App. B

Yielding $f_b = Ve / S_{pl} \leq 0.6F_y$ (2.49)

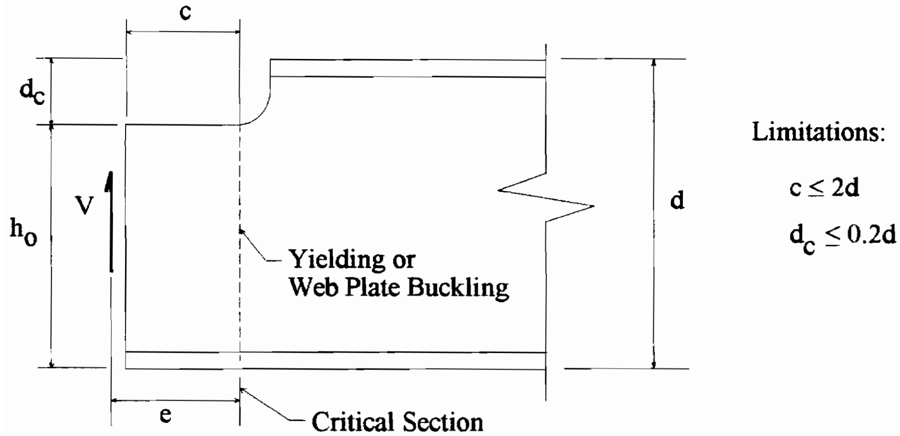
Buckling $f_b = Ve / S_{pl} \leq F_b$ (2.50)

$F_b = 0.6F_{cr}$

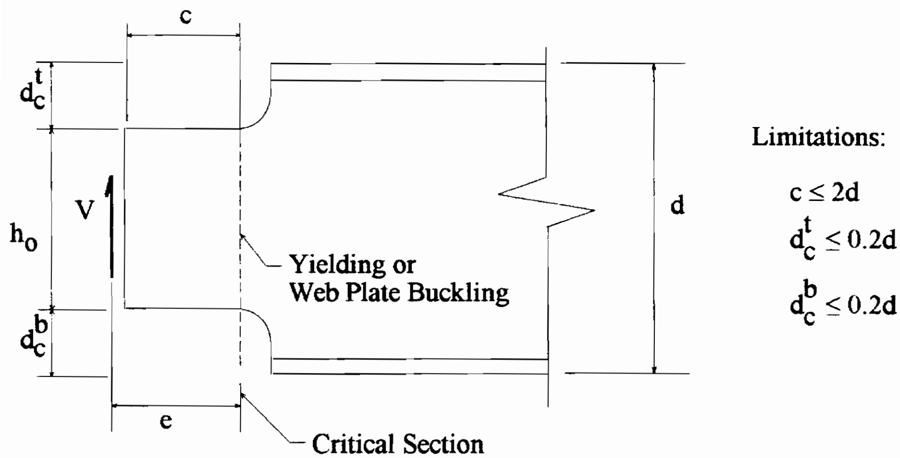
where, F_{cr} same as for LRFD



(a) Bottom Coped Beam



(b) Top Coped Beam



(c) Double Coped Beam

Figure 2.11 Typical Coped Beam Limit States (Murray 1993)

Note: When E = 29,000 ksi,

$$F_{cr} = 33{,}900 \frac{t_w^{\;2}}{ch_o} (3.5 - 7.5\, d_c / d) \qquad (2.51)$$

2.3.12 Local Flange Bending

Beam-to-column strong-axis moment connections transmit either a compressive or tensile force from the beam flange attachment to the column flange. When this force is in tension, it may produce significant deformation in the column flange, thus reducing its capacity. Column flange bending at welded tension flanges can also cause a brittle failure. See Fig. 2.12. The following limit states prevent local column flange bending from occurring.

LRFD: LRFD Spec. K1.2

$$\phi R_n = \phi 6.25 t_f^2 F_{yf} \qquad (2.52)$$

$\phi = 0.90$

If $R_u > \phi R_n$, stiffeners are required. (2.53)

Note: Other requirements exist for bolted tension flange connections (not in Specification)

ASD: ASD Spec. K1.2

$$P_{bf} = 5/3(T_b + T_L) \qquad (2.54)$$

$$= 4/3(T_b + T_L + T_w \text{ or } T_E) \qquad (2.54a)$$

If $t_f \leq 0.4\sqrt{\frac{P_{bf}}{F_{yc}}}$, stiffeners are required. (2.55)

2.3.13 Local Web Yielding

When the transmitted force is compressive to the column as in Fig. 2.13, or perhaps to a beam flange, local yielding of the column or beam web is a concern. Unlike local flange bending, web yielding requirements exist in the AISC Specifications for both bolted and welded compression flange connections. The following calculations are to check the limit state of local web yielding of the column.

LRFD: Eq. 2.53 Applies

Welded LRFD Spec. K1.3

$$R_u \leq \phi R_n = \phi(t_f + 5k)t_w F_y \tag{2.56}$$

$$\phi = 1.0$$

Bolted

$$\phi R_n = \phi F_{yc} t_{wc}(t_{fb} + 6k + 2t_p + 2w) \tag{2.57}$$

$$\phi = 1.0$$

$$w = \text{weld size}$$

ASD:

Welded ASD Spec. K1.9

$$A_{st} = \frac{P_{bf} - F_{yc} t_w (t_{fb} + 5k)}{F_{yst}} \geq 0 \tag{2.58}$$

Bolted ASD 9th ed. p. 4-117

$$P_{bf} = F_{yc} t_{wc}(t_{fb} + 6k + 2t_p + 2w) \tag{2.59}$$

2.3.14 Web Buckling

Another concern with beam-to-column moment connections is buckling of the column web when a compressive force is acting upon it. See Fig. 2.14a. This limit state is particularly a plate buckling problem, with the following limit states:

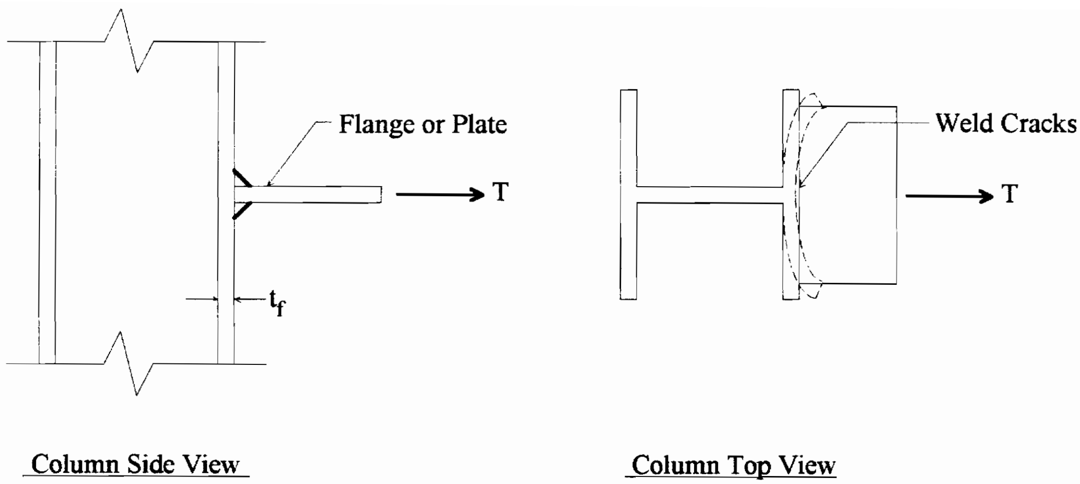


Figure 2.12 Local Flange Bending Limit State

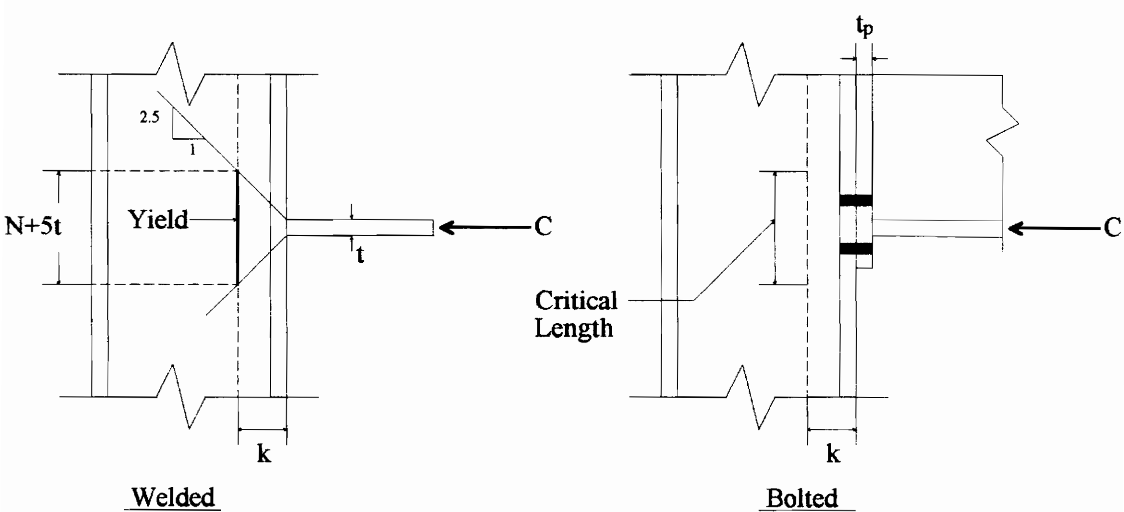
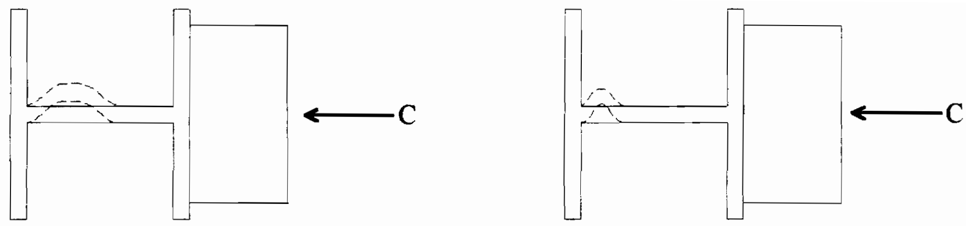


Figure 2.13 Local Web Yielding Limit State



(a) Web Buckling (b) Local Web Crippling

Figure 2.14 Web Buckling and Local Web Crippling Limit States

LRFD: Eq. 2.53 Applies

LRFD Spec. K1.6

$$\phi R_n = \phi \frac{4100 t_{wc}^3 \sqrt{F_{yc}}}{d_c} \tag{2.60}$$

$\phi = 0.90$

d_c = clear distance between fillets

ASD: ASD Spec. K1.6

Stiffeners are required if,

$$d_c > \frac{4100 t_{wc}^3 \sqrt{F_{yc}}}{P_{bf}} \tag{2.61}$$

Note: Stiffeners must be at least (d - t_{fc} - k) deep.

2.3.15 Local Web Crippling

The final limit state associated with beam flange forces acting upon the connecting column is local web crippling. The failure is similar to that of web buckling, but the failure mode is more localized, as seen in Fig. 2.14b. The limit states depend on the location of the compressive force to the column end.

LRFD: Eq. 2.53 Applies

LRFD Spec. K1.4

$$\phi R_n = \phi 135 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw} t_f}{t_w}} \tag{2.62}$$

unless at member end (less than d / 2), then

$$\text{for } N/d \leq 0.2, \phi R_n = \phi 68 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw} t_f}{t_w}} \tag{2.63a}$$

$$\text{for } N/d > 0.2, \phi R_n = \phi 68 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw} t_f}{t_w}} \tag{2.63b}$$

$\phi = 0.75$

ASD: ASD Spec. K1.4

$$R = 67.5t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw}t_f}{t_w}} \tag{2.64}$$

unless at member end (less than d / 2), then

$$R = 34t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw}t_f}{t_w}} \tag{2.65}$$

Note: Stiffeners must be at least ($d - t_{fc} - k$) deep.

2.3.16 Column Web Stiffeners

When any of the limit states from local flange bending (2.3.12) through local web crippling (2.3.15) are violated, strengthening of the column is required. This may be done with stiffeners or doubler plates, depending of the limit state. The following design criteria should be used.

LRFD: LRFD Spec. K1.9

$$w_s + (t_{wc} / 2) \geq b_f / 3 \ \ (\text{or } b_p / 3) \tag{2.66}$$

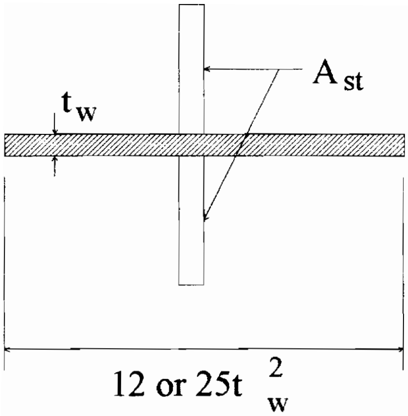
$$t_s \geq t_f / 2 \ \ (\text{or } t_p / 2) \tag{2.67}$$

$$b_s / t_s \ \leq 95 / \sqrt{F_y} \tag{2.68}$$

ASD: ASD Spec. K1.8

Same requirements as for LRFD

If a stiffener is required because of web crippling (K1.4) or web buckling (K1.6), use an effective column section to determine the compressed member capacity (Sect. E.2):



$$A_{equiv} = A_{st} + 25t_w^2 \quad (2.69)$$

(Use $A_{st} + 12t_w^2$ if at column top)

$kL = 0.75h$

$h = d_c - t_{fc} - k$ (if partial depth)

$= d_c - 2k$ (if full depth)

Figure 2.15 Equivalent Stiffener Area

2.3.17 Column Panel Zone Web Shear

Often in rigid beam-to-column connections, the connected beam webs lie in a common plane as shown in Fig. 2.16. When the total required shear strength has exceeded the design shear strength, either double plates or diagonal stiffeners are required (AISC 1993).

LRFD: LRFD Spec. K1.7

For $P_u \le 0.75P_n$

$$\phi R_v = 0.8F_y d_c t_w \quad (2.70)$$

For $P_u > 0.75P_n$

$$\phi R_v = 0.7F_y d_c t_w[1.9 - 1.2(P_u / P_n)] \quad (2.71)$$

$$\phi = 0.9$$

If $R_u > \phi R_v$, double plate(s) are required.

ASD: ASD Spec. K1.7 & F4

$$\Sigma F = \frac{M_1}{0.95d_1} + \frac{M_2}{0.95d_2} - V_s \tag{2.72}$$

$$f_v = \frac{\Sigma F}{\left(d_c t_{wc}\right)} \geq F_v \text{ from Sect. F4} \tag{2.73}$$

If $f_v > F_v$, use double plate(s).

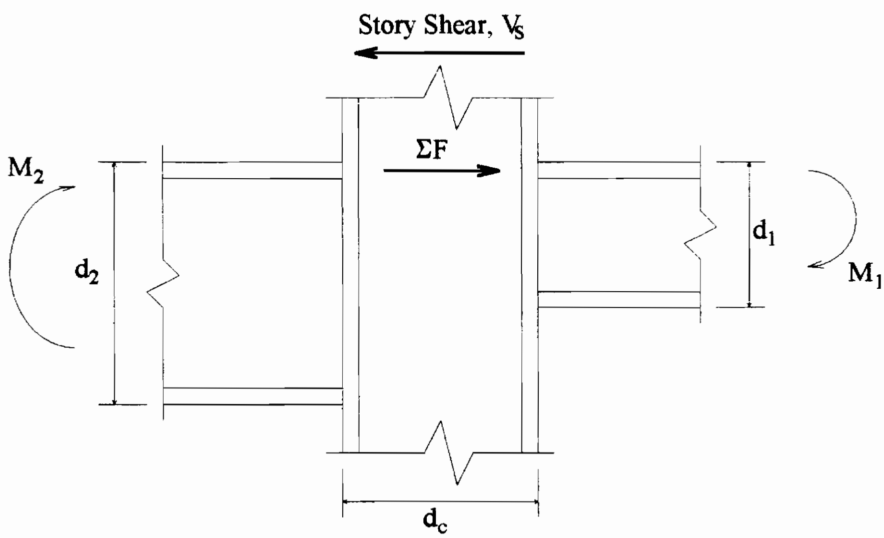


Figure 2.16 Column Panel Zone

2.3.18 Reduced Beam Flexural Strength at Flange Bolted Moment Connection

Frequently, beam-to-column moment connections consist of plates welded to the column and bolted to the beam flanges. If this type of connection is used, the beam's reduced flexural strength, due to the bolt holes, must be examined. The following rules apply for this type of connection.

LRFD: LRFD Spec. B10

No reduction is required if:

$$0.75F_uA_{fn} \geq 0.9F_yA_{fg} \tag{2.74}$$

Else, the effective reduced area of the tension flange becomes:

$$A_{fe} = \frac{0.75F_u}{0.9F_y}A_{fn} = \frac{5}{6}\frac{F_u}{F_y}A_{fn} \tag{2.75}$$

Therefore, member flexural properties are to be calculated with Eq. 2.71. The section properties should be calculated with the removed area from the gross-section. Thus,

$$A_{removed} = A_g - A_{fe} \tag{2.76}$$

Then, a reduced moment capacity may be calculated as follows:

$$M_{p\,reduced} = M_p - [F_yA_{removed}(d - t_f) / 12] \tag{2.77}$$

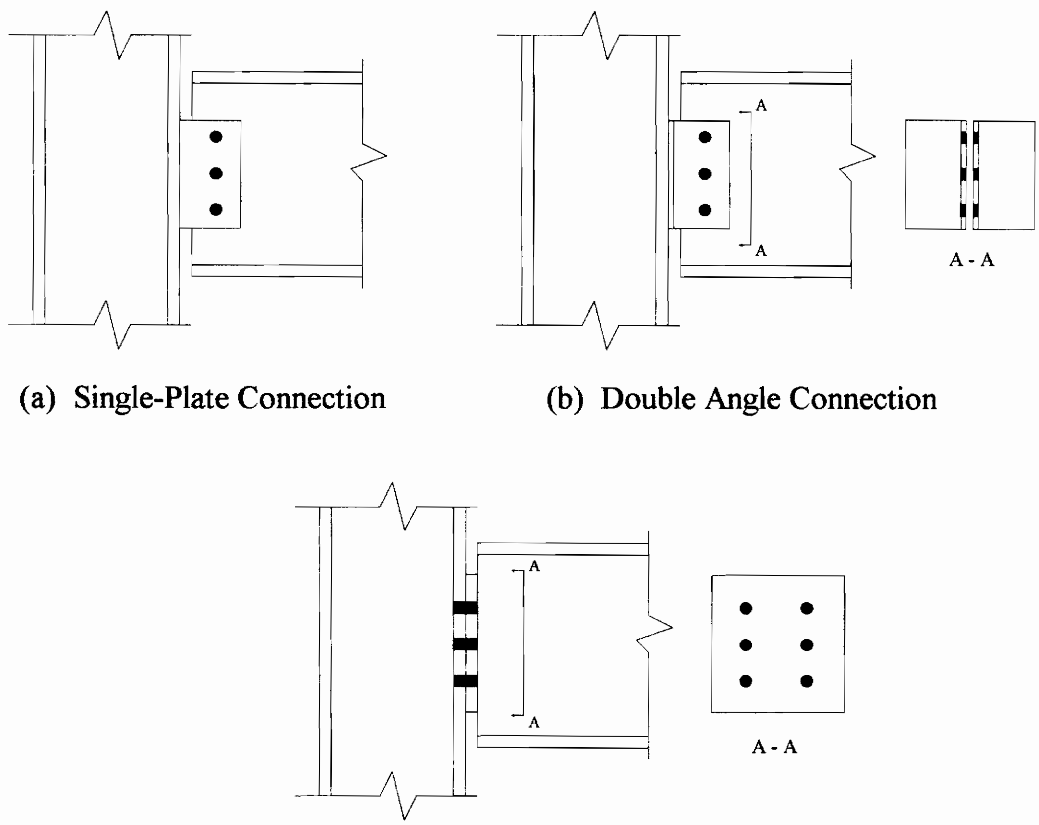
ASD: ASD Spec. B2 & B10

Same rules apply as for LRFD

2.4 Simple Framing Connections

Simple framing connections are classified as PR (LRFD) or Type II (ASD), and are used to connect beams to girders or columns when simple support is assumed. This type of connection assumes the connectors (i.e., plates, angles, tees) are flexible and allow rotation between the two connected elements.

Typically, the column or girder connection is made in the field, while the beam connection is shop fabricated (Salmon and Johnson 1990). These connections may be either welded or bolted, but most often welded connections are done in the shop. Some of the most common simple framing connections are: single-plate, double angle, shear end-plate, single angle, and tee. Fig. 2.17 illustrates some of the most common framing connections.



(a) Single-Plate Connection

(b) Double Angle Connection

(c) Shear End-Plate Connection

Figure 2.17 Typical Simple Framing Connections

With most simple framing connections, the applied load does not pass through the bolt group or weld center of gravity, and the affect of load eccentricity on the fasteners must be addressed. Single-plate framing connections always consider the affect of eccentricity. However, with certain framing angle connections eccentricity may be ignored. Those instances are illustrated in the figures below for both double and single angles (Murray 1993).

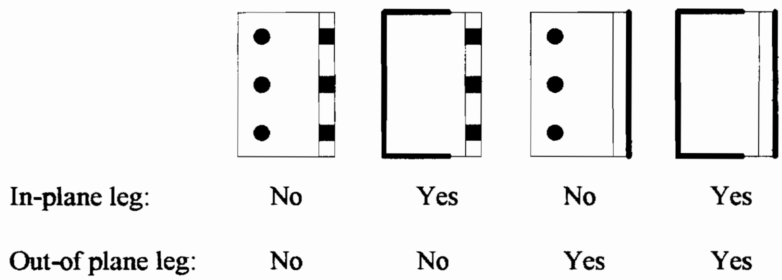


Figure 2.18 Eccentricity Considerations For Double Framing Angles

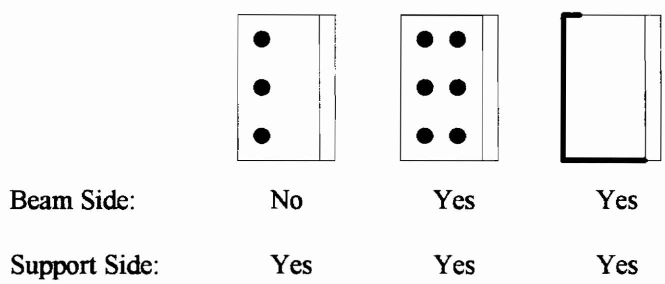
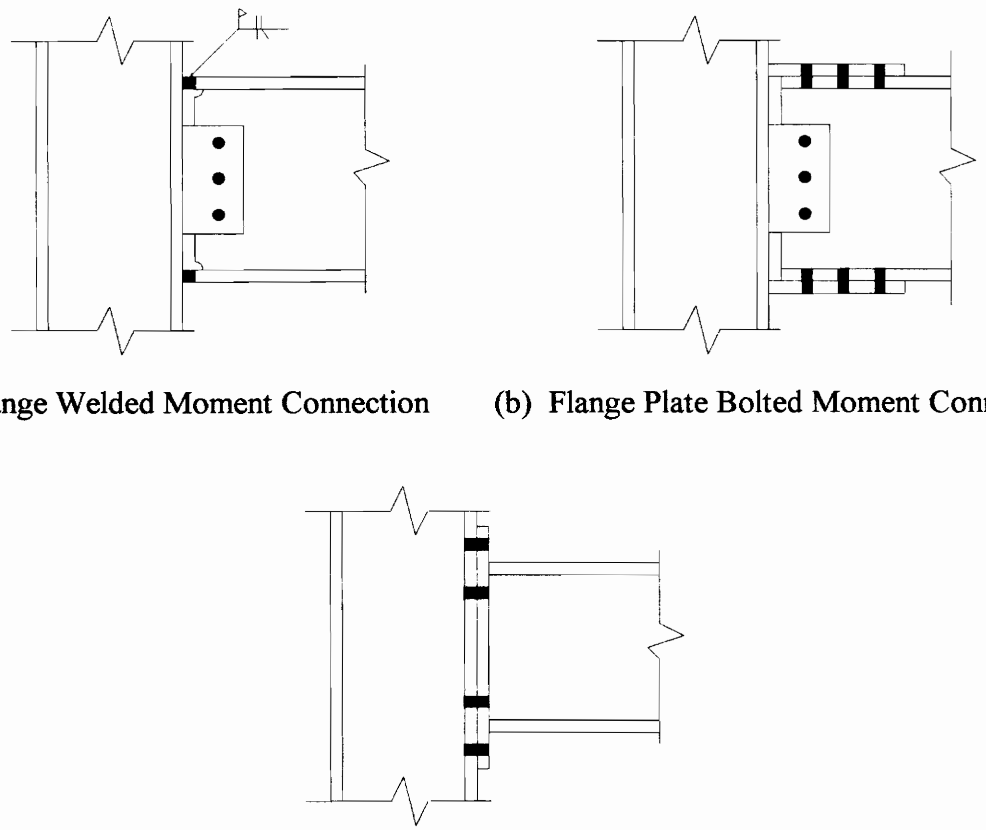


Figure 2.19 Eccentricity Considerations For Single Framing Angles

2.5 Moment Connections

Moment connections are classified as FR (LRFD) or Type I (ASD), and are used to provide a rigid connection between beams and columns, allowing little or no rotation at the joint. This type of connection is used to fully transfer moment between connected members. The flange connections are assumed to transfer all moment, while the web connection transfers nothing but shear. Therefore, unlike in most simple framing connections, the affect of eccentricity on the web connection is not a concern.

The most common moment connections are: flange welded, flange plate, and moment end-plate. Fig. 2.20 below illustrates these connections.



(a) Flange Welded Moment Connection (b) Flange Plate Bolted Moment Connection

(c) Bolted Moment End-Plate Connection

Figure 2.20 Typical Moment Connections

CHAPTER III
SCULPTURE CONNECTION DESIGN

3.1 Steel Sculpture Background

To relay the concepts of steel connection design to the student, the following chapter will cover some of the most common building connections used in industry. Simple framing and moment connections will be covered, as well as column and beam splices and other miscellaneous connections. All of the design examples covered in this report are specifically linked to connections on an actual structure. A basic layout of this structure is represented in Fig. 3.1 through Fig. 3.6. In addition to these figures, shop drawings and erection drawings are provided in the appendices of this report. These drawings will aid in the structure's fabrication and erection, so it may be used for demonstrative purposes in the future.

The original idea for designing a "teaching sculpture" was developed by Dr. Duane Ellifritt, a civil engineering professor at the University of Florida (Ellifritt 1987). This report builds on Dr. Ellifritt's structure by not only giving the students an opportunity to visualize the connections, but by also reviewing the design limit states for each associated connection. This project also aids the professor in his approach to teaching connection design. Therefore, professors and students alike will benefit from this report.

Simple framing connection design examples will be covered first in this chapter, followed by moment connection design. The final section of this chapter will cover limit states of all remaining connections contained in the sculpture that do not fall into the two previous categories (i.e., beam and column splices, bearing connections, truss connections, etc.).

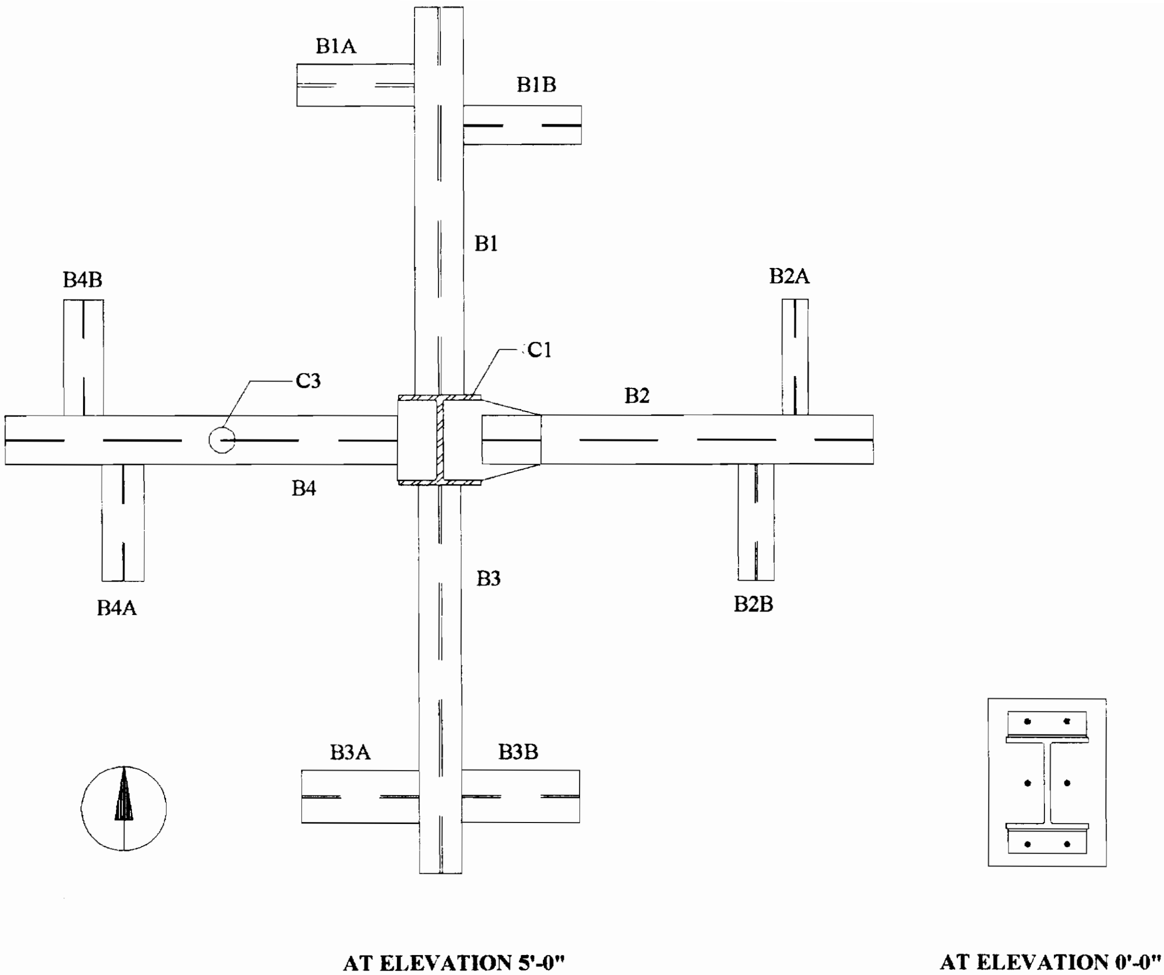


Figure 3.1 Steel Sculpture - Framing Plan

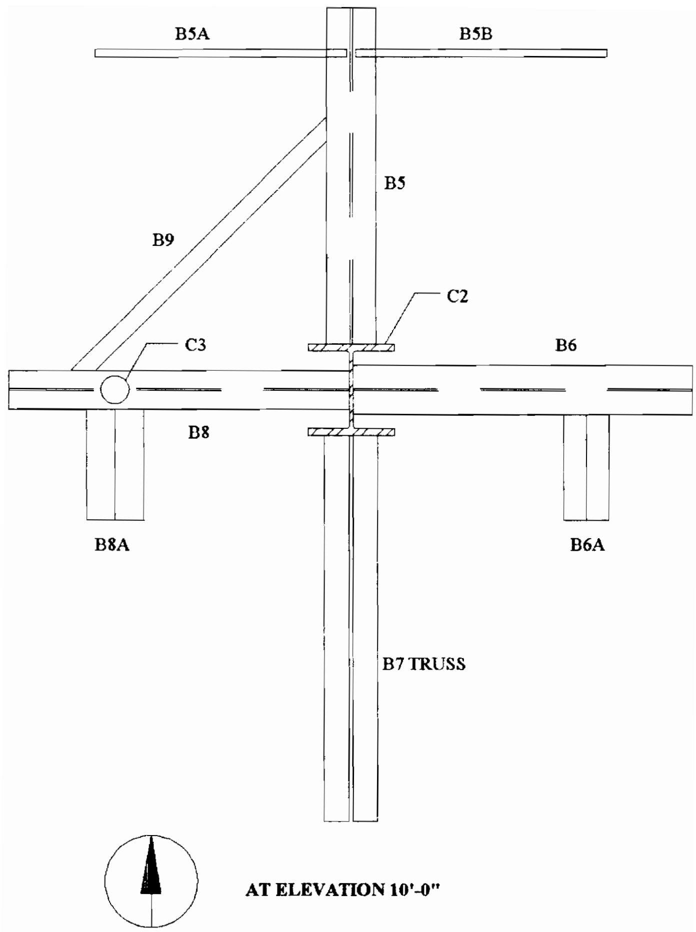


Figure 3.2 Steel Sculpture - Framing Plan

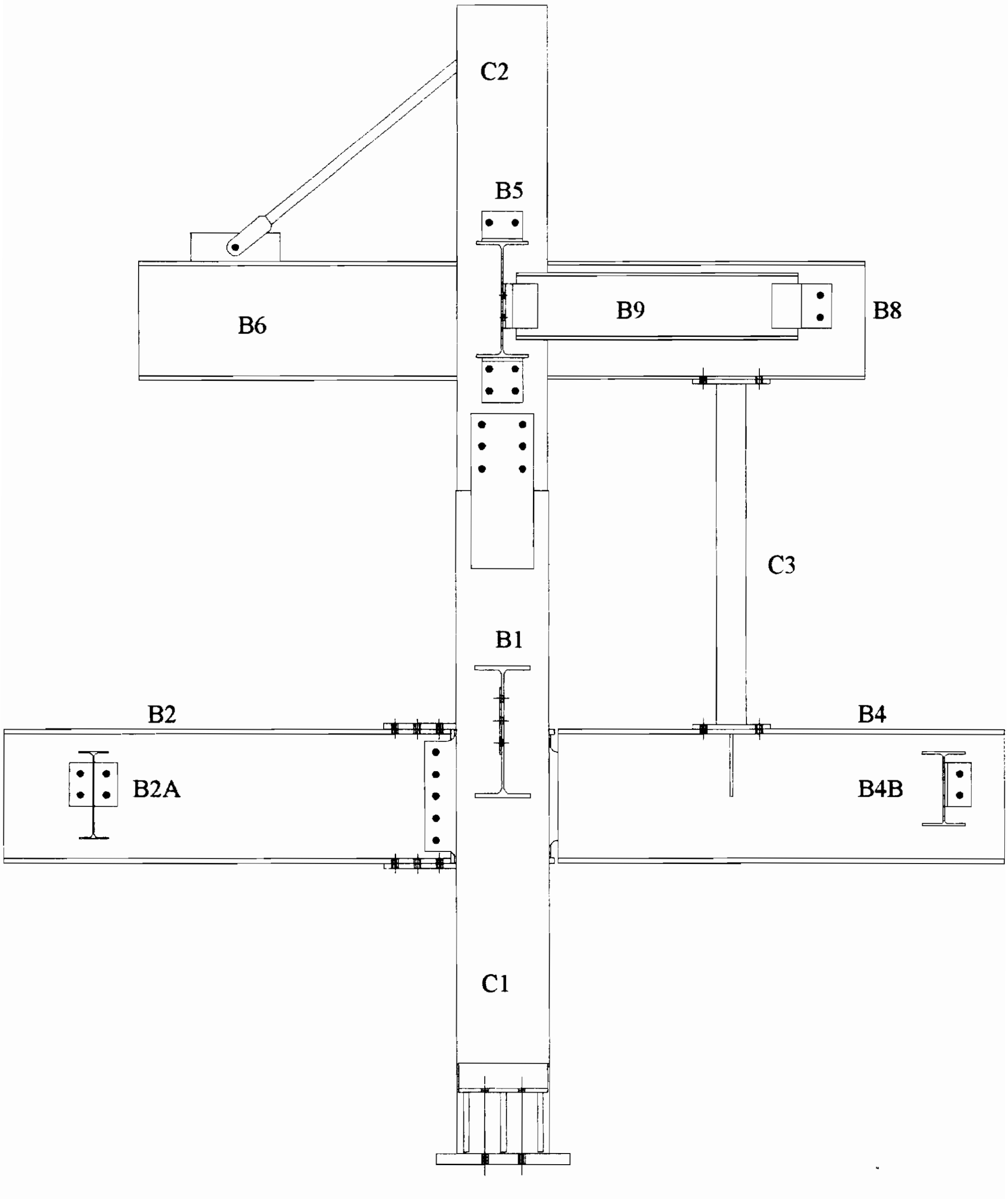


Figure 3.3 Steel Sculpture - North Elevation

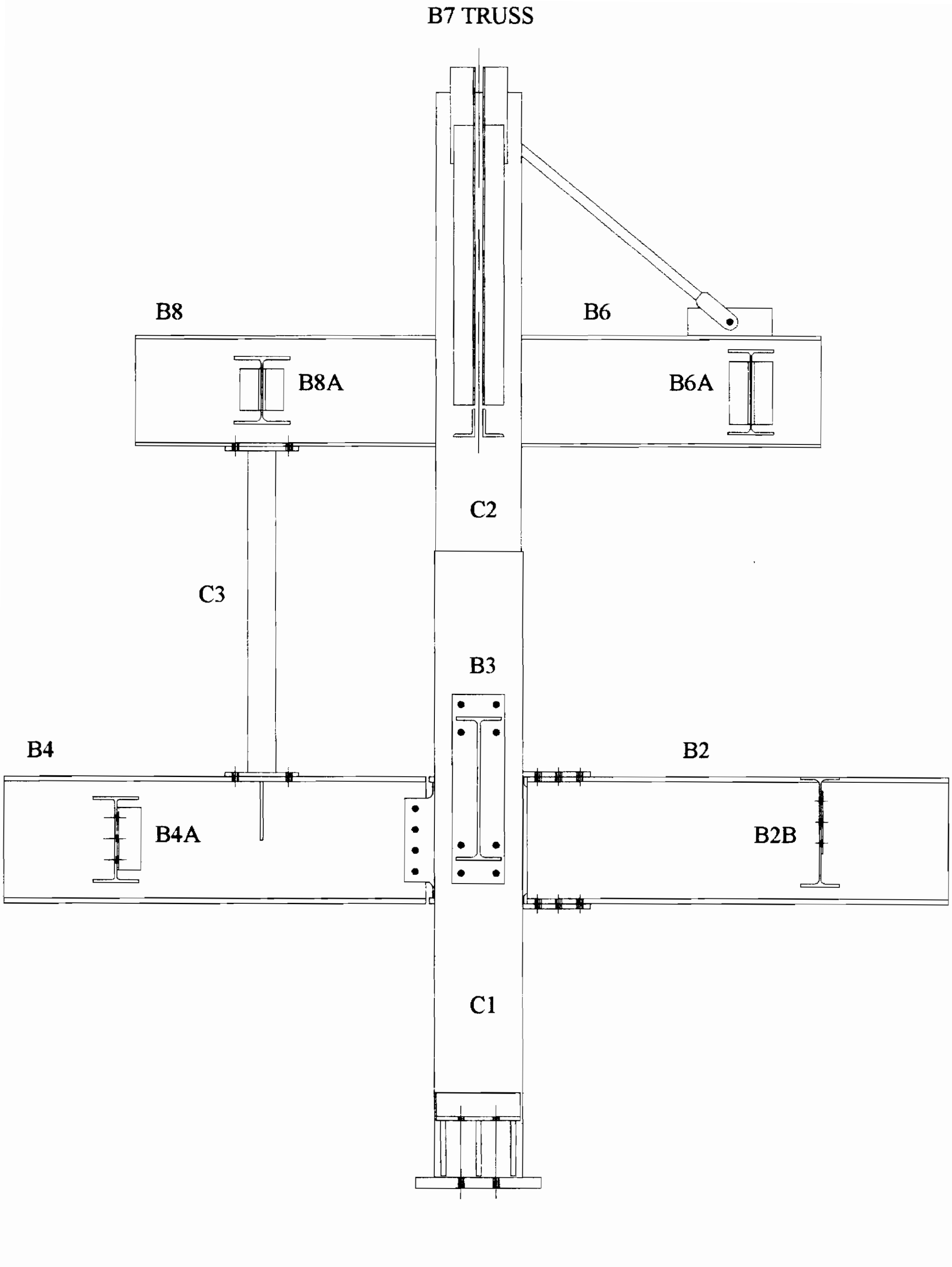


Figure 3.4 Steel Sculpture - South Elevation

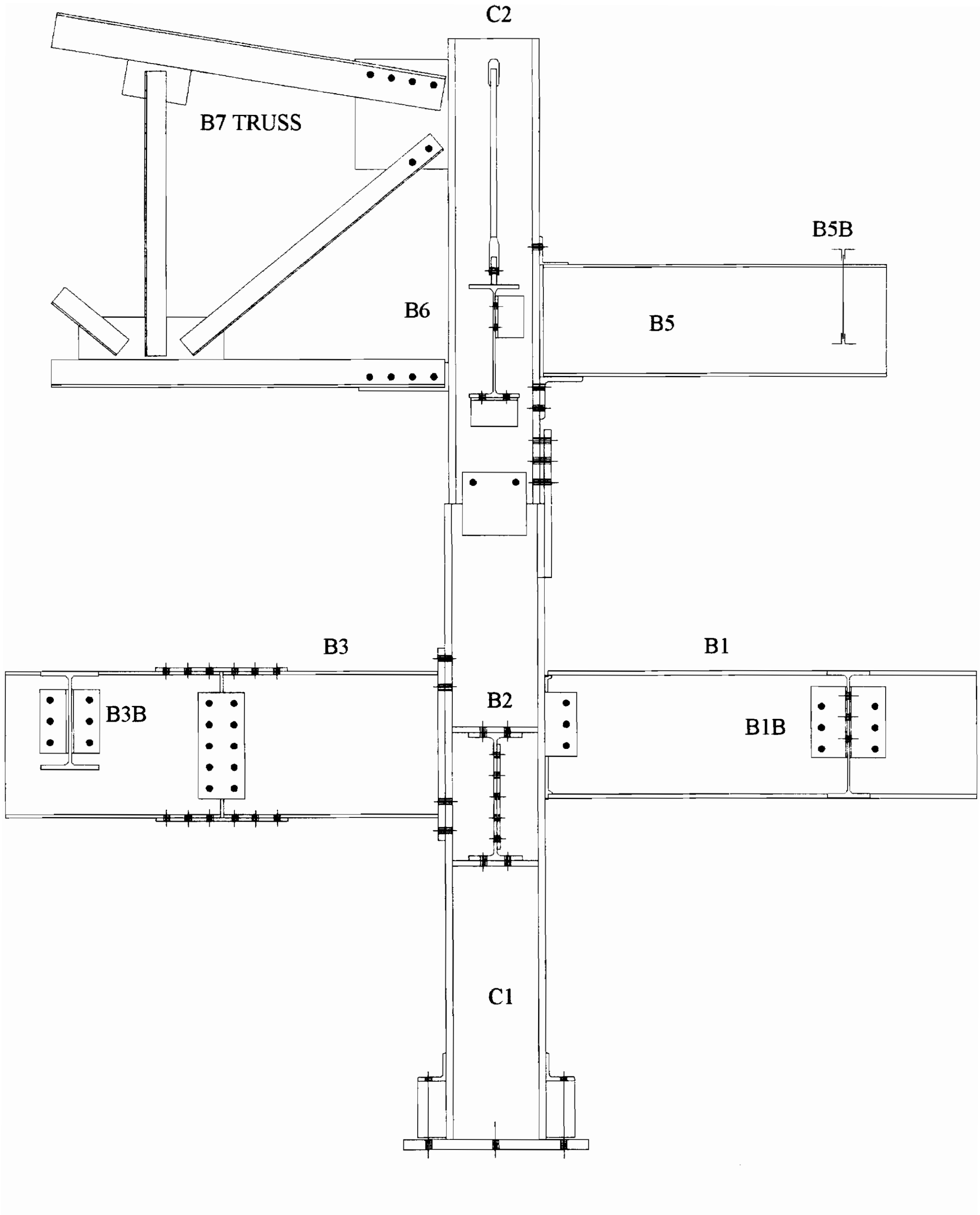


Figure 3.5 Steel Sculpture - East Elevation

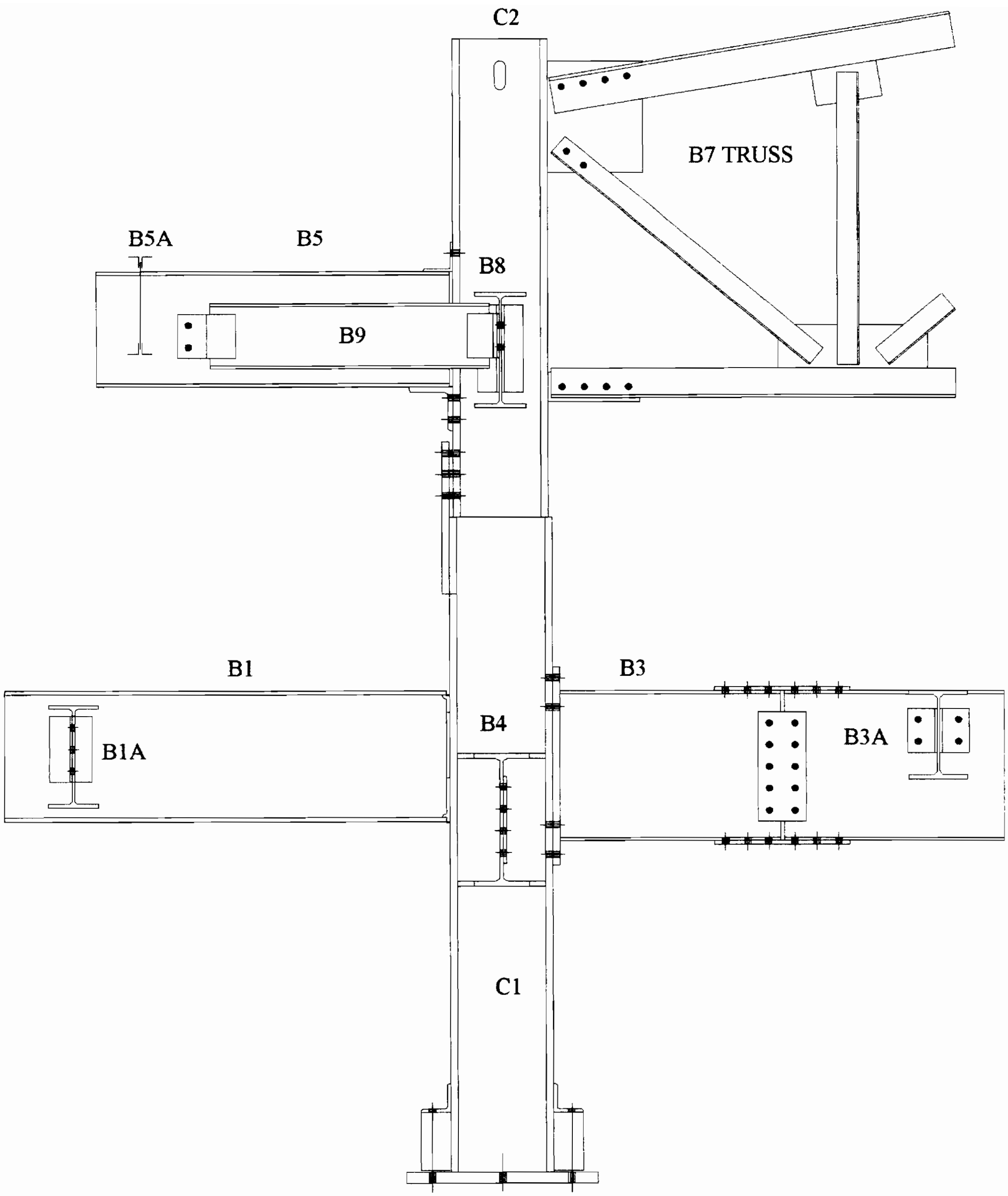


Figure 3.6 Steel Sculpture - West Elevation

3.2 Simple Framing Connection Design Examples

3.2.1 Shear End-Plate Framing Connection

<u>**Example 1.**</u> Refer to connection beam B2A framing into beam B2. (Fig. 3.1, 3.3)

<u>Given:</u> W12x14 beam framing into a W18x60 Girder.
Shear end-plate connection.
PL 1/4x6x6 1/2
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.7 below.

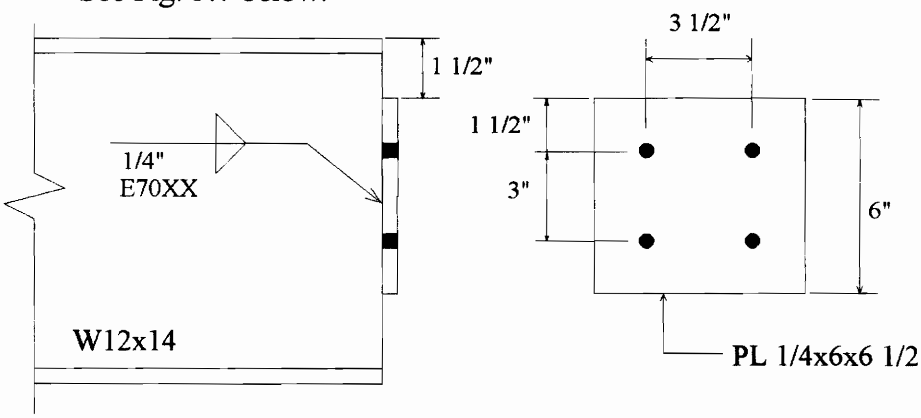


Figure 3.7

<u>Show:</u> All limit states that apply.

<u>Limit States:</u>

1. <u>Beam Gross Shear</u>

$$\phi V_n = \phi(0.6F_yA_w)$$
$$= 0.9(0.6)(36)(11.91)(0.20) = 46.3^k$$

2. <u>Beam Web Strength at Weld</u>

$$\phi V_n = \phi(0.6F_yL_{weld}t_w)$$
$$= 0.9(0.6)(36)(6 - 2(1/4))(0.20)$$
$$= 22.2^k$$

3. Weld Rupture (t_{weld} = 1/4 in)

$$\phi V_n = (1.392^{k/in/16})(\#\text{ of }16_{ths})L_{weld}$$
$$= (1.392 \times 4)(6 - 2(1/4))(2) = 61.2^k$$

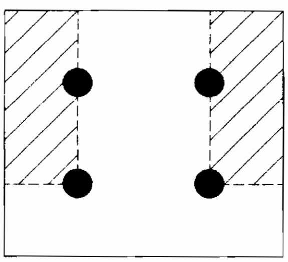
4. Plate Gross Shear

$$\phi V_n = \phi(0.6F_yA_g) = 0.9(0.6)(36)(6 \times .25)(2) = 58.3^k$$

5. Plate Net Shear

$$\phi V_n = \phi(0.6F_uA_{ns}) = 0.75(0.6)(58)(6 - 2(7/8 + 1/8)(1/4 \times 2) = 52.2^k$$

6. Plate Block Shear



Shear Rupture:

$$0.6F_uA_{ns} = 0.6(58)(4.5 - 1.5(7/8 + 1/8))1/4$$
$$= 26.1^k$$

Tension Rupture:

$$F_uA_{nt} = 58(1.5 - 0.5(7/8 + 1/8))1/4$$
$$= 14.5^k$$

Shear Rupture > Tension Rupture

∴ Shear Rupture controls

Tension Yield: $F_yA_{gt} = 36(1.5 \times 1/4) = 13.5^k$

$$\therefore \phi V_n = 0.75(26.1 + 13.5)(2) = 59.4^k$$

7. Plate Bearing / Tear-out

edge = 1.5 in. > $1.5d_b$ & spacing = 3 > $3d_b$

∴ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_ud_bt = 2[2 \times 0.75(2.4)(58)(7/8)(1/4)] = 91.4^k$$

8. Bolt Shear

4 7/8 in. dia. A325-N bolts

$$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(4) = 86.6^k$$

9. <u>Girder Web Bearing / Tear-out</u>
No edge distance concern, spacing = 3 in. > $3d_b$

$\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 2[2 \times 0.75(2.4)(58)(7/8)(.415)] = 151.6^k$$

$\therefore$ <u>Beam Web Strength at Weld controls the connection design</u> $\Rightarrow$ $\phi V_n = 22.2^k$

3.2.2 Single-Plate Framing Connection

Example 1. Refer to connection beam B2B framing into beam B2. (Fig. 3.1, 3.4)

Given: A W16x26 beam framing into a W18x60 Girder.
Single plate framing connection.
PL 1/4x4 1/2x9
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.8 below.

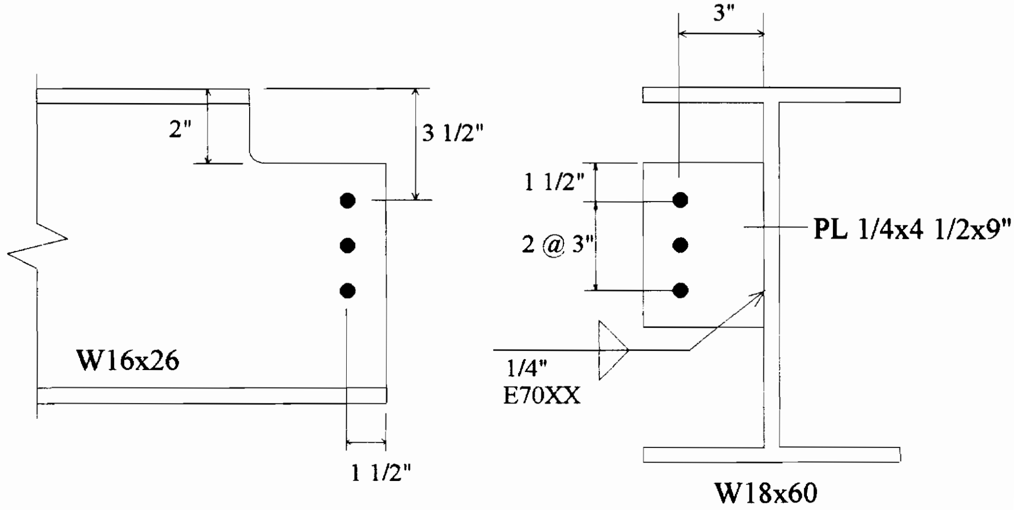


Figure 3.8

Show: All limit states that apply.

Limit States:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6F_yA_w) = 0.9(0.6)(36)(15.69 - 2)(0.25) = 66.5^k$$

2. Beam Net Shear

$$\phi V_n = \phi(0.6F_uA_n) = 0.75(0.6)(58)[15.69 - 2 - 3(7/8 + 1/8)](0.25) = 69.8^k$$

3. Beam Web Bearing/Tear-out
   Must consider eccentricity

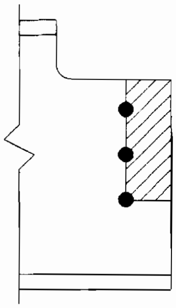
   Using LRFD Table X and the following values:

   $b = 3$ in.
   $x_o = 3$ in. $\quad\therefore\ C = 1.77$ (effective number of bolts)
   $n = 3$

   $e = 1.5$ in. $> 1\ 1/2d_b$ & $s = 3$ in. $> 3d_b$ $\quad\therefore$ Only bearing applies

   $$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 1.77 \times 0.75(2.4)(58)(7/8)(.25) = 40.4^k$$

4. Coped Beam Web Block Shear



Shear Rupture:

$$0.6F_u A_{ns} = 0.6(58)(7.5 - 2.5(7/8 + 1/8))0.25 = 43.5^k$$

Tension Rupture:

$$F_u A_{nt} = 58(1.5 - 0.5(7/8 + 1/8))0.25 = 14.5^k$$

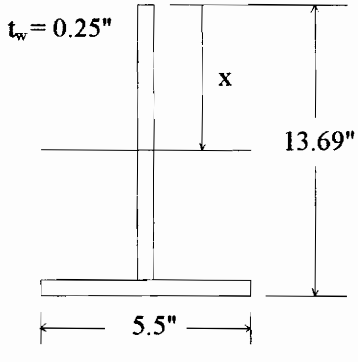
Shear Rupture > Tension Rupture
$\therefore$ Shear Rupture controls

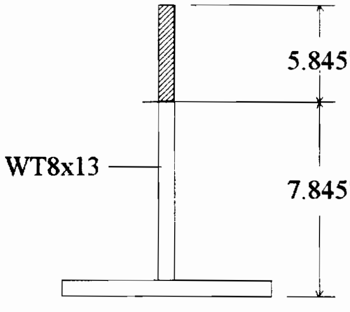
Tension Yield: $F_y A_{gt} = 36(1.5 \times 0.25) = 13.5^k$

$$\therefore\ \phi V_n = 0.75(43.5 + 13.5) = 42.8^k$$

5. Bending Strength of Coped Beam Web

   Consider Structural Tee and Plate





$d_{st} = 7.845$ in.

$$\bar{x} = \frac{\frac{1}{2}(5.845)^2(0.25) + 3.84(5.845 + 7.845 - 2.09)}{(5.845)(0.25) + 3.84}$$

$$\bar{x} = 9.21 \text{ in.}$$

$$I_x = 1/12(0.25)(5.845)^3 + (0.25)(5.845)(9.21 - 5.845/2)^2 + 23.5 + (3.84)(13.69 - 2.09 - 9.21)^2$$

$$I_x = 128.2 \text{ in}^4$$

$$S_t = \frac{128.2}{9.21} = 13.9 \text{ in.}^3$$

Yield: $\phi V_n = \phi F_y S_{tee} / e = 0.9(36)(13.9) / 4 = 112.5^k$

Buckling:

$c / h_o = 2.5 / 13.69 = 0.183 < 1 \quad \therefore \; k = 2.2(h_o / c)^{1.65} = 36.4$

$c / d = 2.5 / 15.69 = 0.16 < 1 \quad \therefore \; f = 2(c / d) = 0.32$

$$\phi V_n = \phi\,[26{,}200\, k\, f\, (t_w / h_o)^2]\,(S_{tee} / e)$$
$$= 0.9[26{,}200(36.4)(0.32)(0.25/13.69)^2](13.9 / 4)$$
$$\phi V_n = 318.3^k$$

$\therefore$ Yield controls $\Rightarrow \phi V_n = 112.5^k$

6. Bolt Shear

4 7/8 in. dia. A325-N bolts, consider eccentricity

$\phi R_b = \phi r_v A_b C = 0.75(48)(0.6013)(1.77) = 38.3^k$

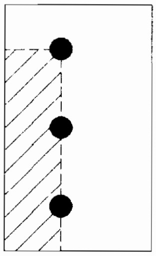
7. Plate Gross Shear

$\phi V_n = \phi(0.6 F_y A_g) = 0.9(0.6)(36)(9 \text{ x } .25) = 43.7^k$

8. Plate Net Shear

$\phi V_n = \phi(0.6 F_u A_{ns}) = 0.75(0.6)(58)(9 - 3(7/8 + 1/8)0.25 = 39.2^k$

9. Plate Block Shear



Shear Rupture:

$0.6F_uA_{ns} = 0.6(58)(7.5 - 2.5(7/8 + 1/8))0.25$
$= 43.5^k$

Tension Rupture:

$F_uA_{nt} = 58(1.5 - 0.5(7/8 + 1/8))0.25$
$= 14.5^k$

Shear Rupture > Tension Rupture
∴ Shear Rupture controls

Tension Yield: $F_yA_{gt} = 36(1.5 \times 0.25) = 13.5^k$

$\therefore\ \phi V_n = 0.75(43.5 + 13.5) = 42.8^k$

7. Plate Bearing / Tear-out
edge > $1.5d_b$ & spacing > $3d_b$ ∴ Only bearing applies

Note: Use effective number of bolts, C = 1.77

$\phi R_n = \phi \sum_{bolts} 2.4F_ud_bt = 1.77[0.75(2.4)(58)(7/8)(.25)] = 40.4^k$

8. Weld Rupture at Girder Web

For a flexible support, $e_w = n - 1 = 3 - 1 = 2$ in.

Using LRFD Table XVIII and the following values:

$al = e_w = 2$ in ⇒ $a = 0.22$
$kl = 0$ ⇒ $k = 0$

Interpolation gives, C = 2.184

$\therefore\ \phi R_n = P_u = C_1CDl = 1.0(2.184)(4)(9) = 78.6^k$

9. Girder Web Shear at Weld

$\phi R_n = 0.9(0.6)(36)(9 \times 0.415)(2) = 145.2^k$

∴ Bolt Shear controls the connection design ⇒ $\phi V_n = 38.3^k$

3.2.3 Double Framing Angle Connection

Example 1. Refer to connection beam B1B framing into beam B1. (Fig. 3.1, 3.5)

Given: A W18x40 beam framing into a W18x50 Girder.
Double angle connection.
2L-SLBB 5x3x1/4. Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.9 below.

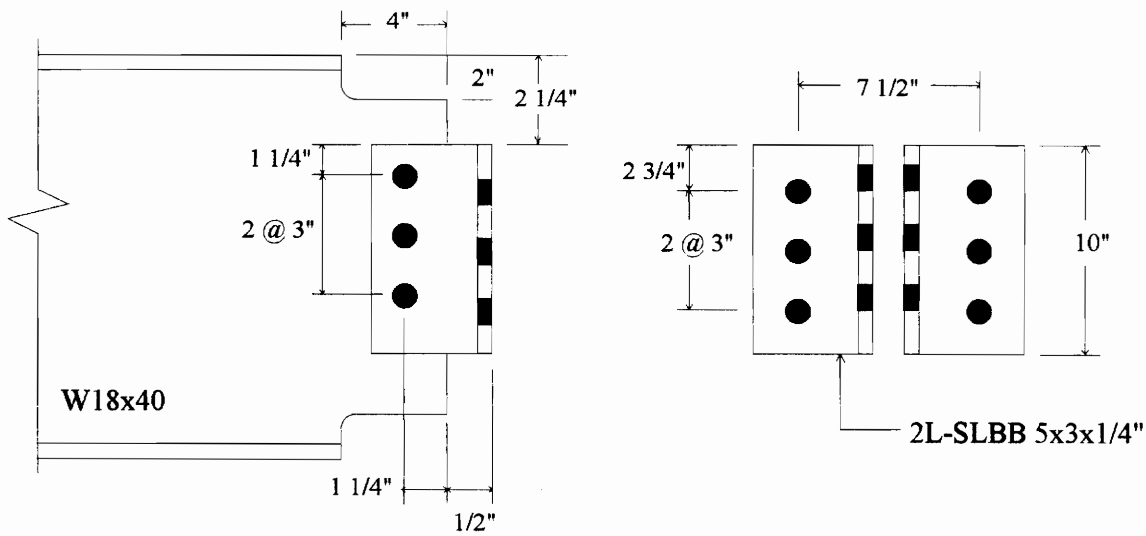


Figure 3.9

Show: All limit states that apply.

Limit States:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6\, F_y\, A_w) = 0.9(0.6)(36)(17.9 - 2(2))(0.315) = 85.1^k$$

2. Beam Net Shear

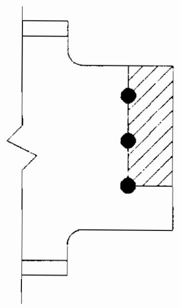
$$\phi V_n = \phi(0.6 F_u A_n) = 0.75(0.6)(58)[17.9 - 2(2) - 3(7/8 + 1/8)](0.315) = 89.6^k$$

3. Beam Web Bearing/Tear-out

$e = 1.5$ in. $> 1\ 1/2 d_b$ & $s = 3$ in. $> 3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 0.75(3)(2.4)(58)(7/8)(.315) = 86.3^k$$

4. Coped Beam Web Block Shear



Shear Rupture:

$$0.6F_uA_{ns} = 0.6(58)(7.5 - 2.5(7/8 + 1/8))0.315$$
$$= 54.8^k$$

Tension Rupture:

$$F_uA_{nt} = 58(1.75 - 0.5(7/8 + 1/8))0.315$$
$$= 22.8^k$$

Shear Rupture > Tension Rupture

∴ Shear Rupture controls

Tension Yield: $F_yA_{gt} = 36(1.34 \times 0.315) = 15.2^k$

$$\therefore\ \phi V_n = 0.75(54.8 + 15.2) = 52.5^k$$

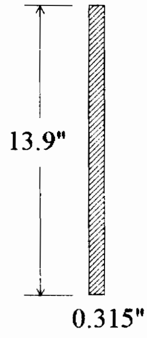
5. Bolt Shear

6 7/8 in. dia. A325-N bolts, eccentricity is ignored.

$$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(3)(2) = 129.6^k$$

6. Bending Strength of Coped Beam Web

In this case, treat as a plate



$A = 4.3875 \text{ in}^2$ $\qquad$ $I_x = 1/12(0.315)(13.9)^3 = 70.5 \text{ in}^4$

$\bar{y} = 6.95$ in. $\qquad$ $S_{pl} = \dfrac{I_x}{\bar{y}} = 10.14 \text{ in.}^3$

Yield: $\qquad e \approx 4 + 1.75 = 5.75$ in.

$$\phi R_n = \phi F_y S_{pl} / e = 0.9(36)(10.14) / 5.75 = 57.1^k$$

Buckling: $\qquad F_{cr} = 56{,}500 \dfrac{t_w}{ch_o}^2 (3.5 - 7.5\, d_c / d)$

$$F_{cr} = 56{,}500 \frac{(.315)}{4(13.9)}^2 [3.5 - 7.5\,(2/17.9)] = 268.4 \text{ ksi}$$

∴ Since $F_y << F_{cr}$, Yielding controls $\Rightarrow$ $\phi V_n = 57.1^k$

7. Angle Bearing / Tear-out

edge = 1.25 in. $< 1.5d_b$, spacing = 3 $> 3d_b$ $\therefore$ Tear-out also applies

for exterior bolts:

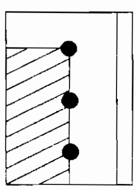
$$R_n = \min \left| \begin{array}{l} F_u e t = 58(1.25)(0.25) = 18.1^k \\ 2.4\, F_u d_b t = 2.4(58)(0.875)(0.25) = 30.45^k \end{array} \right.$$

for interior bolts:

$$R_n = \min \left| \begin{array}{l} F_u (s - d/2)t = 58(3 - 0.875/2)(0.25) = 37.16^k \\ 2.4\, F_u d_b t = 2.4(58)(0.875)(0.25) = 30.45^k \end{array} \right.$$

$$\therefore \ \phi R_n = 0.75[1(18.1) + 2(30.45)](2) = 118.5^k$$

8. Angle Block Shear



Shear Rupture:

$$0.6F_u A_{ns} = 0.6(58)(8.75 - 2.5(7/8 + 1/8))0.25 = 54.4^k$$

Tension Rupture:

$$F_u A_{nt} = 58(1.25 - 0.5(7/8 + 1/8))(0.25) = 10.9^k$$

Shear Rupture > Tension Rupture
$\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(1.25 \times 1/4) = 11.25^k$

$$\therefore \ \phi V_n = 0.75(54.4 + 11.25)(2) = 98.4^k$$

9. Angle Net Shear

$$\phi V_n = \phi(0.6F_u A_{ns}) = 0.75(0.6)(58)[10 - 3(7/8 + 1/8)](1/4)(2) = 91.3^k$$

10. Angle Gross Shear

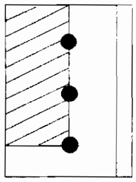
$$\phi V_n = \phi(0.6F_y A_g) = 0.9(0.6)(36)(10 \times 0.25)(2) = 97.2^k$$

11. Angle Bearing/Tear-out (Girder Side)

edge = 2.75 in. $> 1.5d_b$, spacing = 3 $> 3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = (2)(3)[(0.75)(2.4)(58)(7/8)(1/4)] = 137^k$$

12. Angle Block Shear (Girder Side)



Shear Rupture:

$$0.6F_uA_{ns} = 0.6(58)(8.75 - 2.5(7/8 + 1/8))0.25 = 54.4^k$$

Tension Rupture:

$$F_uA_{nt} = 58(1.4 - 0.5(7/8 + 1/8))(0.25) = 10.15^k$$

Shear Rupture > Tension Rupture
$\therefore$ Shear Rupture controls

Tension Yield: $F_yA_{gt} = 36(1.4 \text{ x } 1/4) = 12.6^k$

$$\therefore \ \phi V_n = 0.75(54.4 + 12.6)(2) = 100.5^k$$

13. Bolt Shear
3 7/8 in. dia. A325-N bolts, eccentricity is ignored.

$$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(3)(2) = 129.9^k$$

14. Girder Web Bearing / Tear-out
No edge distance concern, spacing = 3 in. > $3d_b$ $\quad\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = (2)(3)[(0.75)(2.4)(58)(7/8)(.355)] = 194.6^k$$

$\therefore$ Block Shear at Coped Beam Web controls the connection design $\Rightarrow$ $\phi V_n = 52.5^k$

Example 2. Refer to connection beam B3A framing into beam B3. (Fig. 3.1, 3.6)

Given: A W12x50 beam framing into a W21x44 Girder.
Double angle connection.
2L-SLBB 4x3 1/2x3/8. Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.10 below.

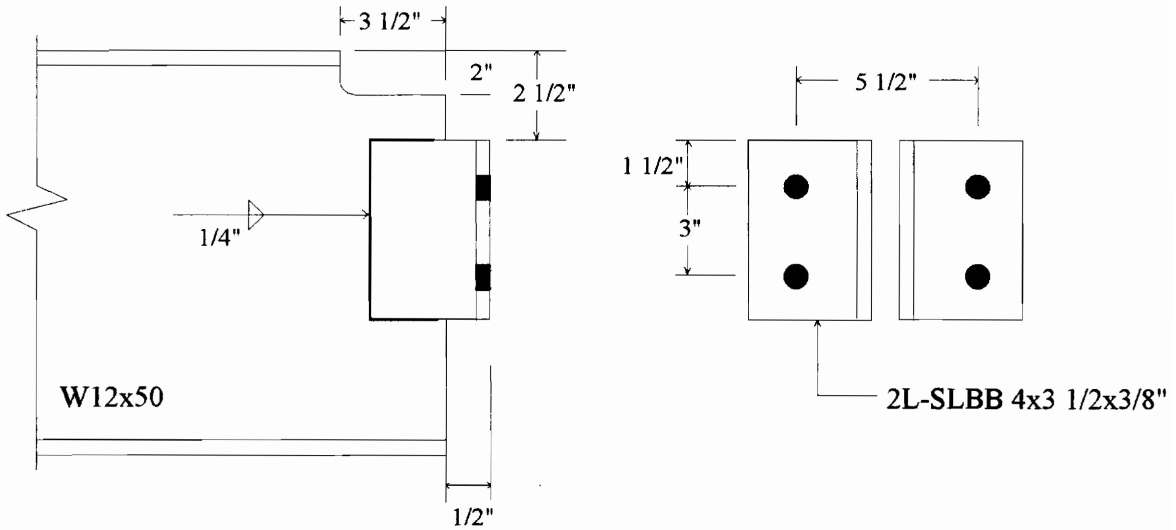


Figure 3.10

Show: All limit states that apply.

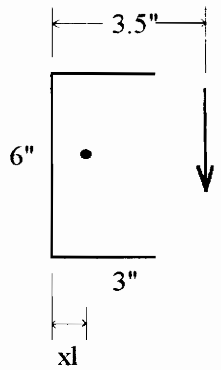
Limit States:

1. Beam Gross Shear

$$\begin{aligned}\phi V_n &= \phi(0.6\,F_y\,A_w)\\ &= 0.9(0.6)(36)[12.19 - 2](0.37)\\ &= 73.3^k\end{aligned}$$

2. Weld Shear Strength ($t_{weld} = 1/4$ in)

Using LRFD Table XXII and the following values:



$$xl = \frac{2[3(1.5)]}{6+2(3)} = 0.75 \text{ in.}$$

$$al = 3.5 \text{ in.} - xl = 2.75 \text{ in.}$$

$$a = \frac{al}{l} = \frac{2.75 \text{ in.}}{6 \text{ in.}} = 0.46$$

$$k = \frac{kl}{l} = \frac{3 \text{ in.}}{6 \text{ in.}} = 0.5$$

Interpolation gives C = 1.815

$$\therefore \phi R_n = P_u = C_1 C D l = (1)(1.815)(4)(6)\text{x}2 = 87.2^{k}$$

3. Beam Web Strength at Weld

Yield strength per inch of beam $= \phi(0.6F_y t_w)$

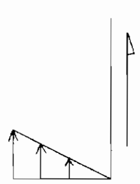
$$= 0.9(0.6)(36)(0.37) = 7.19^{k/in.}$$

Strength per inch of weld $= 1.392(2 \times 4) = 11.14^{k/in.}$

$7.19 < 11.14$ $\therefore$ web is not adequate for full weld

$$\frac{7.19}{11.14}(87.2) = 56.3 \text{ k}$$

4. Block Shear at Coped Beam Web



$$\phi V_n = \phi[0.6F_y l t_w + (F_u / 2) a t_w]$$

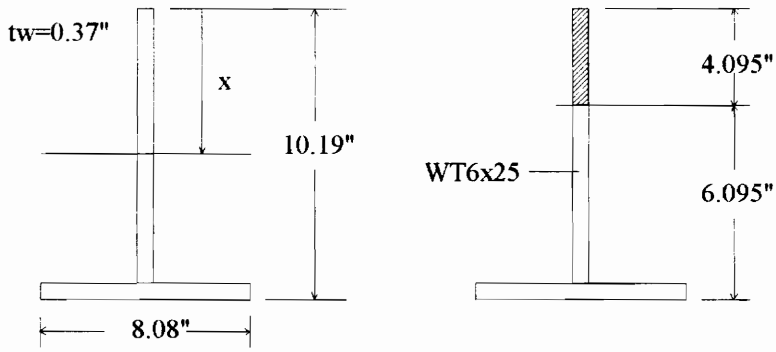
$$= 0.75[0.6(36)(6)(0.37) + (58/2)(2.75)(0.37)]$$

$$= 58.1^{k}$$

Note: Cope less than in-plane angle leg

5. Bending Strength of Coped Beam Web

Consider Structural Tee and Plate



$$\bar{x} = \frac{\frac{1}{2}(4.095)^2(0.37) + 7.34(10.19 - 1.17)}{(4.095)(0.37) + 7.34}$$

$$\bar{x} = 7.83 \text{ in.}$$

$$I_x = 1/12(0.37)(4.095)^3 + (0.37)(4.095)(7.83 - 4.095/2)^2 + 18.7 + (7.34)(10.19 - 1.17 - 7.83)^2$$

$$I_x = 81.9 \text{ in}^4$$

$$S_{tee} = \frac{81.9}{7.83} = 10.46 \text{ in.}^3$$

Yield: $F_y = 36$ ksi

Buckling:

$c / h_o = 3.5 / 10.19 = 0.34 < 1 \quad \therefore\ k = 2.2(h_o / c)^{1.65} = 12.83$

$c / d = 3.5 / 12.19 = 0.287 < 1 \quad \therefore\ f = 2(c / d) = 0.574$

$$F_{cr} = 26{,}200\, k\, f\, (t_w / h_o)^2 = 26{,}200(12.83)(0.574)(0.37/10.19)^2 = 254.4 \text{ ksi}$$

$\therefore$ Since $F_{cr} >> F_y \quad \Rightarrow \quad$ Yield controls

$$\therefore\ \phi V_n = \phi F_y S_{tee} / e = 0.9(36)(10.46) / 4 = 84.7^k$$

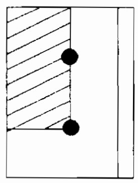
6. Angle Gross Shear

$$\phi V_n = \phi(0.6 F_y A_g) = 0.9(0.6)(36)(6 \times 0.375)(2) = 87.5^k$$

7. Angle Net Shear

$$\phi V_n = \phi(0.6F_uA_{ns}) = 0.75(0.6)(58)(6 - 2(7/8 + 1/8)(0.375)(2) = 78.3^k$$

8. Angle Block Shear



Shear Rupture:

$$0.6F_uA_{ns} = 0.6(58)[4.5 - 1.5(7/8 + 1/8)](0.375) = 39.15^k$$

Tension Rupture:

$$F_uA_{nt} = 58[1.435 - 0.5(7/8 + 1/8)](0.375) = 20.33^k$$

Shear Rupture > Tension Rupture

$\therefore$ Shear Rupture controls

Tension Yield: $F_yA_{gt} = 36(1.435 \times 0.375) = 19.37^k$

$$\therefore\ \phi V_n = [0.75(39.15 + 19.37)](2) = 87.8^k$$

9. Angle Bearing/Tear-out
edge > $1.5d_b$, spacing $3d_b$ $\quad\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_ud_bt = (2)(2)[(0.75)(2.4)(58)(7/8)(.375)] = 137^k$$

10. Bolt Shear
4 7/8 in. dia. A325-N bolts, eccentricity is ignored.

$$\phi R_b = \phi r_vA_bN_b = 0.75(48)(0.6013)(4) = 86.6^k$$

11. Girder Web Bearing / Tear-out
No edge distance concern, spacing = 3 in. > $3d_b$ $\quad\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_ud_bt = (2)(2)[(0.75)(2.4)(58)(7/8)(.35)] = 127.9^k$$

$\therefore$ Beam Web Strength at Weld controls the connection design $\Rightarrow$ $\phi V_n = 56.3^k$

Example 3. Refer to connection beam B3B framing into beam B3. (Fig. 3.1, 3.5)

Given: A W12x50 beam framing into a W21x44 Girder.
Double angle connection.
2L-SLBB 4x3 1/2x3/8. Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.11 below.

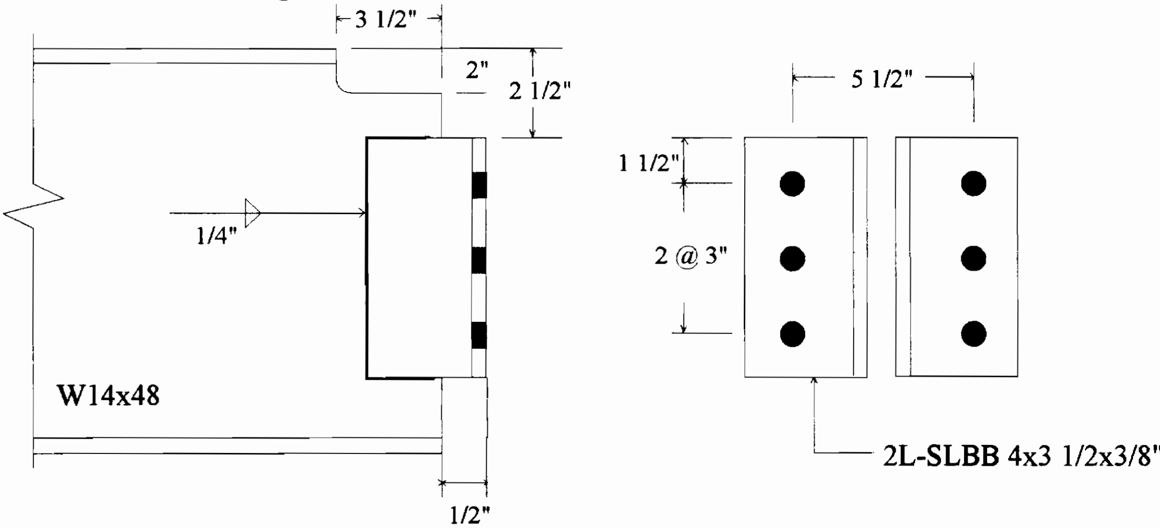


Figure 3.11

Show: All limit states that apply.

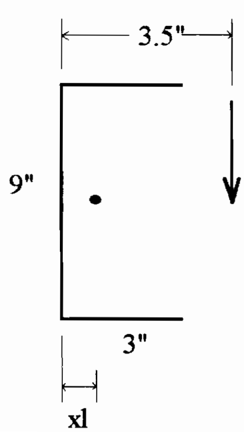
Limit States:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6\ F_y\ A_w)$$
$$= 0.9(0.6)(36)[13.79 - 2](0.34)$$
$$= 77.9^k$$

2. Weld Shear Strength (t_{weld} = 1/4 in)

Using LRFD Table XXII and the following values:



$$xl = \frac{2[3(1.5)]}{9 + 2(3)} = 0.6 \text{ in.}$$

$$al = 3.5 \text{ in.} - xl = 2.9 \text{ in.}$$

$$a = \frac{al}{l} = \frac{2.9 \text{ in.}}{9 \text{ in.}} = 0.32$$

$$k = \frac{kl}{l} = \frac{3 \text{ in.}}{9 \text{ in.}} = 0.33$$

Interpolation gives C = 1.753

$$\therefore \ \phi R_n = P_u = C_1 CDl = (1)(1.753)(4)(9)x2 = 126.2^k$$

3. Beam Web Strength at Weld

Yield strength per inch of beam $= \phi(0.6F_y t_w)$

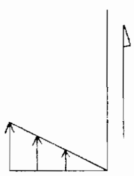
$$= 0.9(0.6)(36)(0.34) = 6.61^{k/in.}$$

Strength per inch of weld $= 1.392(2 \times 4) = 11.14^{k/in.}$

$6.61 < 11.14$ $\therefore$ web is not adequate for full weld

$$\frac{6.61}{11.14}(126.2) = 74.9 \text{ k}$$

4. Block Shear at Coped Beam Web



$$\phi V_n = \phi[0.6F_y l t_w + (F_u / 2) a t_w]$$

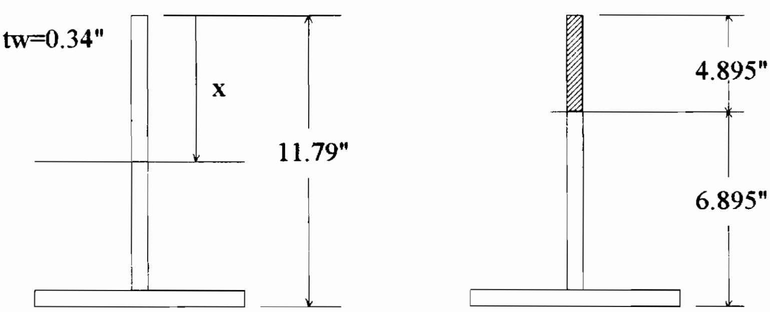
$$= 0.75[0.6(36)(9)(0.34) + (58/2)(2.75)(0.34)]$$

$$= 69.9^k$$

Note: Cope less than in-plane angle leg

5. Bending Strength of Coped Beam Web

Consider Structural Tee and Plate



$$\bar{x} = \frac{\frac{1}{2}(4.895)^2(0.34) + 7.07(11.79 - 1.35)}{(4.895)(0.34) + 7.07}$$

$\bar{x} = 8.92$ in.

$I_x = 1/12(0.3)(4.895)^3 + (0.34)(4.895)(8.92 - 4.895/2)^2$
$\quad + 24.09 + (7.07)(11.79 - 1.35 - 8.92)^2$

$I_x = 113.47 \text{ in}^4$

$$S_{tee} = \frac{113.47}{8.92} = 12.72 \text{ in.}^3$$

Yield: $F_y = 36$ ksi

Buckling:

$c / h_o = 3.5 / 11.79 < 1 \quad \therefore \; k = 2.2(h_o / c)^{1.65} = 16.32$
$c / d = 3.5 / 13.79 < 1 \quad \therefore \; f = 2(c / d) = 0.51$

$F_{cr} = 26{,}200\, k\, f\, (t_w / h_o)^2$
$= 26{,}200(16.32)(0.51)(0.34/11.79)^2$
$= 181.3$ ksi

$\therefore$ Since $F_{cr} >> F_y \quad \Rightarrow \quad$ Yield controls

$\therefore \; \phi V_n = \phi F_y S_{tee} / e = 0.9(36)(12.72) / 4 = 103^k$

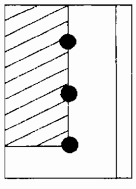
6. Angle Gross Shear

$\phi V_n = \phi(0.6 F_y A_g) = 0.9(0.6)(36)(9 \times 0.375)(2) = 131.2^k$

7. Angle Net Shear

$\phi V_n = \phi(0.6 F_u A_{ns}) = 0.75(0.6)(58)[9 - 3(7/8 + 1/8)](0.375)(2) = 117.5^k$

8. Angle Block Shear



Shear Rupture:

$0.6 F_u A_{ns} = 0.6(58)[7.5 - 2.5(7/8 + 1/8)](0.375) = 65.25^k$

Tension Rupture:

$F_u A_{nt} = 58[1.435 - 0.5(7/8 + 1/8)](0.375) = 20.33^k$

Shear Rupture > Tension Rupture
$\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(1.435 \times 0.375) = 19.37^k$

$\therefore \; \phi V_n = [0.75(65.25 + 19.37)](2) = 126.9^k$

9. Angle Bearing/Tear-out

edge $> 1.5d_b$, spacing $3d_b$ $\quad\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = (2)(3)[(0.75)(2.4)(58)(7/8)(.375)] = 205.5^k$$

10. Bolt Shear

6 7/8 in. dia. A325-N bolts, eccentricity is ignored.

$$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(6) = 129.9^k$$

11. Girder Web Bearing / Tear-out

No edge distance concern, spacing $= 3$ in. $> 3d_b$ $\quad\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = (2)(3)[(0.75)(2.4)(58)(7/8)(.35)] = 191.8^k$$

$\therefore$ Block Shear at Coped Beam controls the connection design $\Rightarrow$ $\phi V_n = 69.9^k$

Example 4. Refer to connection beam B6A framing into beam B6. (Fig. 3.2, 3.4)

Given: A W12x26 beam framing into a W16x40 Girder.
Double angle connection.
2L-SLBB 3x3x3/8. Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.12 below.

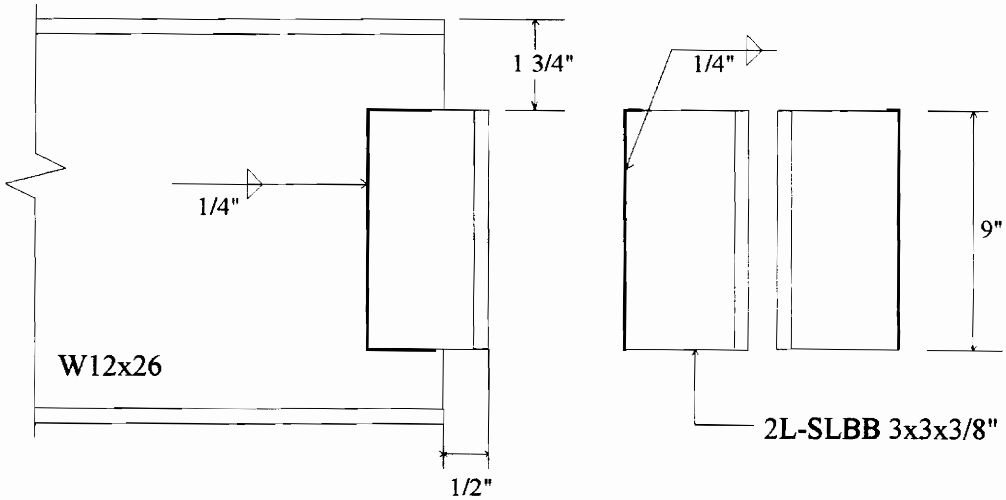


Figure 3.12

Show: All limit states that apply.

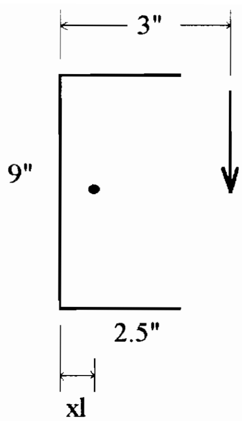
Limit States:

1. Beam Gross Shear

$$\begin{aligned}\phi V_n &= \phi(0.6\, F_y\, A_w) \\ &= 0.9(0.6)(36)(12.22)(0.23) \\ &= 54.6^k\end{aligned}$$

2. Weld Shear Strength (t_{weld} = 1/4 in)

Using LRFD Table XXII and the following values:



$$xl = \frac{2[2.5(1.25)]}{9 + 2(2.5)} = 0.45 \text{ in.}$$

$$al = 3 \text{ in.} - xl = 2.55 \text{ in.}$$

$$a = \frac{al}{l} = \frac{2.55 \text{ in.}}{9 \text{ in.}} = 0.28$$

$$k = \frac{kl}{l} = \frac{2.5 \text{ in.}}{9 \text{ in.}} = 0.28$$

Interpolation gives C = 1.84

$$\therefore \phi R_n = P_u = C_1 CDl = (1)(1.84)(4)(6)x2 = 132.5^k$$

3. Beam Web Strength at Weld

Yield strength per inch of beam $= \phi(0.6F_y t_w)$

$$= 0.9(0.6)(36)(0.23) = 4.47^{k/in.}$$

Strength per inch of weld $= 1.392(2 \text{ x } 4) = 11.14^{k/in.}$

$4.47 < 11.14$ $\therefore$ web is not adequate for full weld

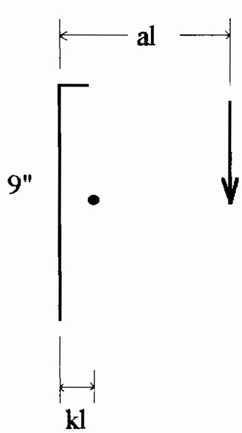
$$\frac{4.47}{11.14}(132.5) = 53.2 \text{ k}$$

4. Angle Gross Shear

$$\phi V_n = \phi(0.6F_y A_g) = 0.9(0.6)(36)(9 \text{ x } 0.375)(2) = 131.2^k$$

5. Weld Shear Strength ($t_{weld} = 1/4$ in)

Using LRFD Table XXIV and the following values:



Neglecting returns ⇒ $kl = 0$, $k = 0$

$$al = 3 \text{ in.} + \frac{0.23}{2} = 3.115 \text{ in.}$$

$$a = \frac{al}{l} = \frac{3.115 \text{ in.}}{9 \text{ in.}} = 0.35$$

Interpolation gives C = 0.86

$$\therefore \phi R_n = P_u = C_1 CDl = (1)(0.86)(4)(9)x2 = 61.9^k$$

6. <u>Shear Yield of Girder Web at Weld</u>:

$$\phi R_n = 0.9(0.6)(36)(9 \times 0.305)(2) = 106.7^k$$

$\therefore$ <u>Beam Strength at Weld controls the connection design</u> $\Rightarrow$ $\phi V_n = 53.2^k$

Example 5. Refer to connection beam B8A framing into beam B8. (Fig. 3.2, 3.4)

Given: A W10x33 beam framing into a W16x31 Girder.
Double angle connection.
2L-SLBB 3x3x3/8. Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.13 below.

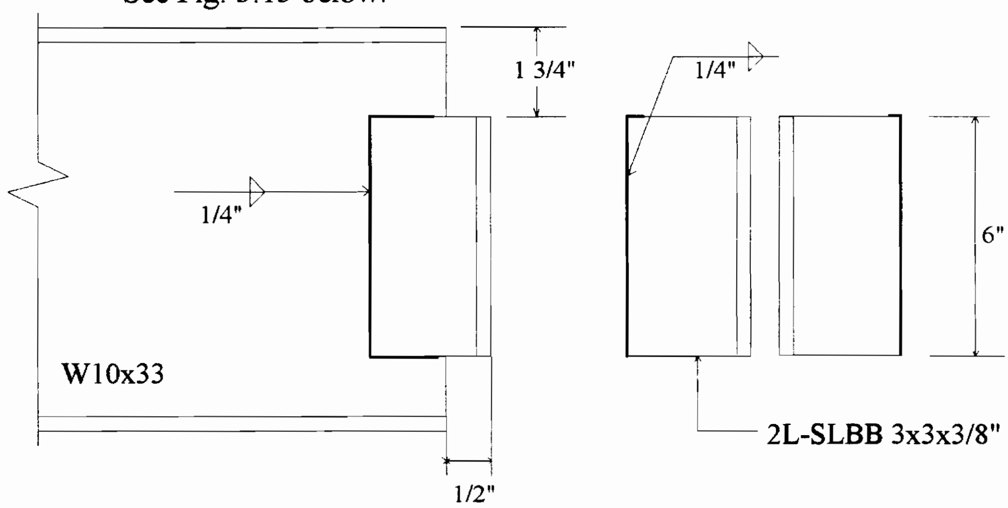


Figure 3.13

Show: All limit states that apply.

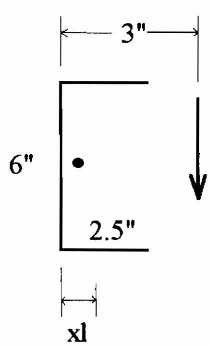
Limit States:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6\, F_y\, A_w)$$
$$= 0.9(0.6)(36)(9.73)(0.29)$$
$$= 54.9^k$$

2. Weld Shear Strength (t_{weld} = 1/4 in)

Using LRFD Table XXII and the following values:



$$xl = \frac{2[2.5(1.25)]}{6 + 2(2.5)} = 0.57 \text{ in.}$$

$$al = 3 \text{ in.} - xl = 2.43 \text{ in.}$$

$$a = \frac{al}{l} = \frac{2.43 \text{ in.}}{6 \text{ in.}} = 0.40$$

$$k = \frac{kl}{l} = \frac{2.5 \text{ in.}}{6 \text{ in.}} = 0.42$$

Interpolation gives C = 1.77

$\therefore\ \phi R_n = P_u = C_1 C D l = (1)(1.77)(4)(6)\text{x}2 = 85.0^k$

3. Beam Web Strength at Weld

Yield strength per inch of beam $= \phi(0.6F_y t_w)$

$= 0.9(0.6)(36)(0.29) = 5.64^{k/in.}$

Strength per inch of weld $= 1.392(2 \text{ x } 4) = 11.14^{k/in.}$

$5.64 < 11.14$ $\therefore$ web is not adequate for full weld

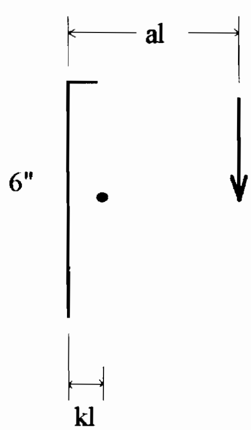
$$\frac{5.64}{11.14}(85) = 43^k$$

4. Angle Gross Shear

$\phi V_n = \phi(0.6F_y A_g) = 0.9(0.6)(36)(6 \text{ x } 0.375)(2) = 87.5^k$

5. Weld Shear Strength (t_{weld} = 1/4 in)

Using LRFD Table XXIV and the following values:



Neglecting returns $\Rightarrow$ $kl = 0$, $k = 0$

$$al = 3 \text{ in.} + \frac{0.29}{2} = 3.145 \text{ in.}$$

$$a = \frac{al}{l} = \frac{3.145 \text{ in.}}{6 \text{ in.}} = 0.52$$

Interpolation gives C = 0.63

$\therefore\ \phi R_n = P_u = C_1 C D l = (1)(0.63)(4)(6)\text{x}2 = 30.2^k$

6. Shear Yield of Girder Web at Weld:

$\phi R_n = 0.9(0.6)(36)(6 \text{ x } 0.275)(2) = 64.2^k$

$\therefore$ Weld Rupture (Girder side) controls the connection design $\Rightarrow$ $\phi V_n = 30.2^k$

Example 6. Refer to connection beam B8 framing into column C2. (Fig. 3.2, 3.6)

Given: A W16x31 beam framing into a W12x106 Column.
Double angle connection.
2L-SLBB 3x3x3/8. Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.14 below.

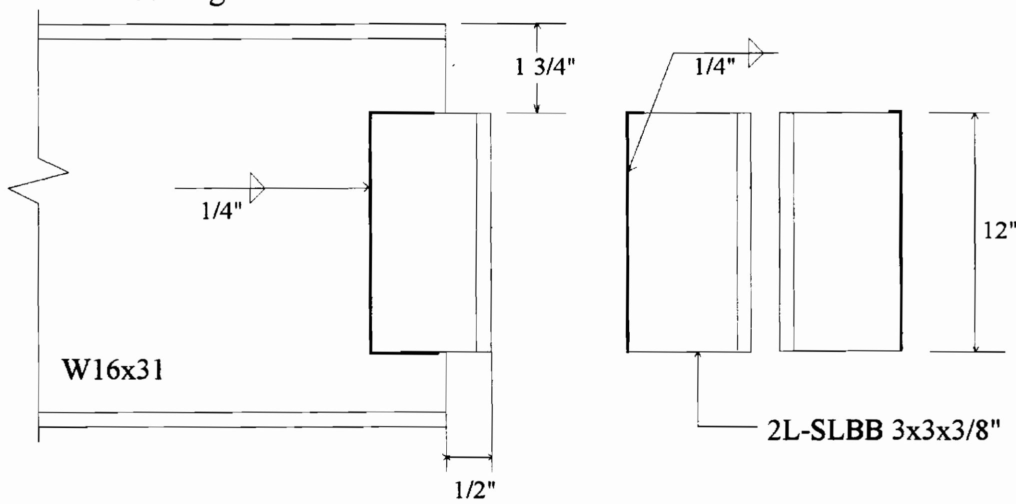


Figure 3.14

Show: All limit states that apply.

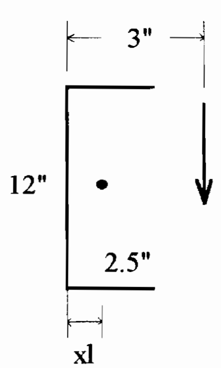
Limit States:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6\, F_y\, A_w)$$
$$= 0.9(0.6)(36)(15.88 \times 0.275)$$
$$= 84.9^k$$

2. Weld Shear Strength ($t_{weld} = 1/4$ in)

Using LRFD Table XXII and the following values:



$$xl = \frac{2[2.5(1.25)]}{12 + 2(2.5)} = 0.37 \text{ in.}$$

$$al = 3 \text{ in.} - xl = 2.63 \text{ in.}$$

$$a = \frac{al}{l} = \frac{2.63 \text{ in.}}{12 \text{ in.}} = 0.22$$

$$k = \frac{kl}{l} = \frac{2.5 \text{ in.}}{12 \text{ in.}} = 0.21$$

Interpolation gives C = 1.68

$\therefore\ \phi R_n = P_u = C_1 C D l = (1)(1.68)(4)(12)x2 = 161.3^k$

3. <u>Beam Web Strength at Weld</u>

Yield strength per inch of beam $= \phi(0.6 F_y t_w)$

$= 0.9(0.6)(36)(0.275) = 5.35^{k/in.}$

Strength per inch of weld $= 1.392(2 \times 4) = 11.14^{k/in.}$

$5.35 < 11.14$ $\therefore$ web is not adequate for full weld

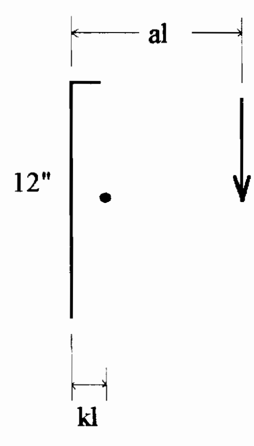
$$\frac{5.35}{11.14}(161.3) = 77.5^k$$

4. <u>Angle Gross Shear</u>

$\phi V_n = \phi(0.6 F_y A_g) = 0.9(0.6)(36)(12 \times 0.375)(2) = 175^k$

5. <u>Weld Shear Strength</u> (t_{weld} = 1/4 in)

Using LRFD Table XXIV and the following values:



Neglecting returns $\Rightarrow$ $kl = 0$, $k = 0$

$$al = 3\text{ in.} + \frac{0.275}{2} = 3.1375\text{ in.}$$

$$a = \frac{al}{l} = \frac{3.1375\text{ in.}}{12\text{ in.}} = 0.26$$

Interpolation gives C = 1.013

$\therefore\ \phi R_n = P_u = C_1 C D l = (1)(1.013)(4)(12)x2 = 97.2^k$

6. <u>Shear Yield of Column Web at Weld</u>:

$\phi R_n = 0.9(0.6)(36)(12 \times 0.61)(2) = 284.6^k$

$\therefore$ <u>Beam Strength at Weld controls the connection design</u> $\Rightarrow$ $\phi V_n = 77.5^k$

3.2.4 Single Framing Angle Connection

Example 1. Refer to connection beam B4A framing into girder B4. (Fig. 3.1, 3.4)

Given: A W12x26 beam framing into a W18x60 Girder.
Single angle connection.
L4x3 1/2x3/8. Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.15.

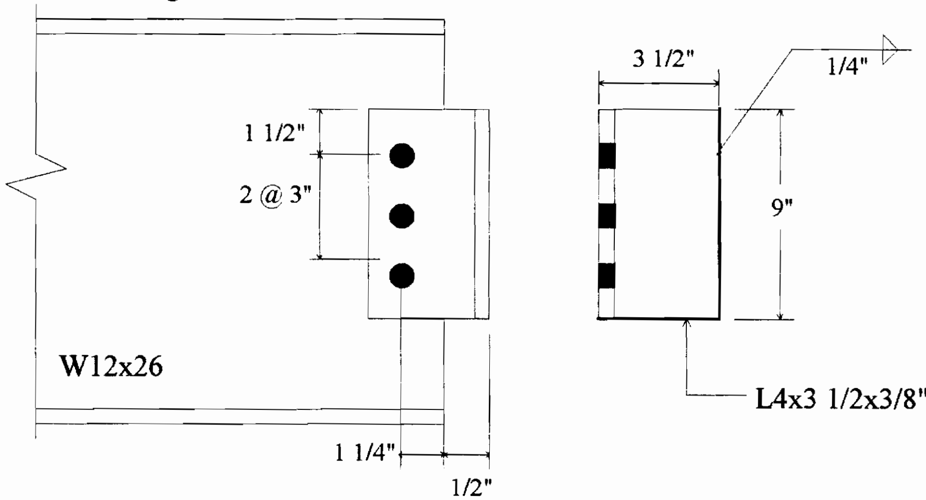


Figure 3.15

Show: All limit states that apply.

Limit States:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6\,F_y\,A_w)$$
$$= 0.9(0.6)(36)(12.22 \times 0.23)$$
$$= 54.6^k$$

2. Beam Net Shear

$$\phi V_n = \phi(0.6 F_u A_n) = 0.75(0.6)(58)[12.22 - 3(7/8 + 1/8)](0.23) = 55.3^k$$

3. Beam Web Bearing/Tear-out

No edge distance concern, s = 3 in. > $3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 0.75(3)(2.4)(58)(7/8)(.23) = 63^k$$

4. Bolt Shear

3 7/8 in. dia. A325-N bolts, eccentricity is ignored.

$$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(3) = 64.9^k$$

5. Angle Gross Shear

$$\phi V_n = \phi(0.6F_y A_g) = 0.9(0.6)(36)(9 \times 0.375) = 65.6^k$$

6. Angle Net Shear

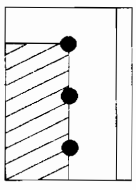
$$\phi V_n = \phi(0.6F_u A_{ns}) = 0.75(0.6)(58)[9 - 3(7/8 + 1/8)](3/8) = 58.7^k$$

7. Angle Bearing/Tear-out (Girder Side)

edge $> 1.5d_b$, spacing $> 3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = (3)[(0.75)(2.4)(58)(7/8)(3/8)] = 102.8^k$$

8. Angle Block Shear



Shear Rupture:

$$0.6F_u A_{ns} = 0.6(58)[7.5 - 2.5(7/8 + 1/8)](0.375) = 65.25^k$$

Tension Rupture:

$$F_u A_{nt} = 58(2.25 - 0.5(7/8 + 1/8))(0.375) = 38.1^k$$

Shear Rupture > Tension Rupture

$\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(2.25 \times 3/8) = 30.4^k$

$$\therefore \phi V_n = [0.75(65.25 + 30.4)] = 71.7^k$$

9. Flexural Yield of Angle:

$$\phi V_n = 0.9F_y S_g / e, \qquad S_g = \frac{l^2 t}{6} = \frac{(9)^2(3/8)}{6} = 5.06 \text{ in}^2$$

$$\therefore \phi V_n = \frac{0.9(36)(5.06)}{\left(1.75 + \dfrac{0.415}{2}\right)} = 83.8^k$$

10. Flexural Rupture of Angle:

$$\phi V_n = 0.75 F_y S_{net} / e$$

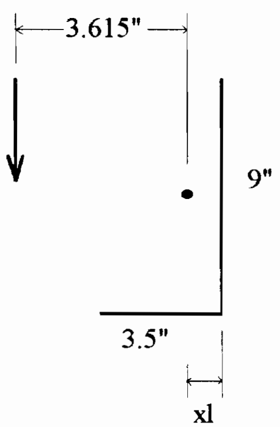
$$S_{net} = S_g - \frac{s^2 N_b (N_b{}^2 - 1)[t(d_b + 1/8)]}{6l}$$

$$= 5.06 - \frac{(3)^2(3)(3^2 - 1)[0.375(7/8 + 1/8)]}{6(9)} = 3.56 \text{ in}^3$$

$$\therefore \phi V_n = \frac{0.75(58)(3.56)}{\left(1.75 + \frac{0.415}{2}\right)} = 79.1^k$$

11. Weld Shear Strength (t_{weld} = 1/4 in)

Using LRFD Table XXIV and the following values:



$$xl = \frac{3.5(1.75)}{3.5 + 9} = 0.49 \text{ in.}$$

$$al = 3.615 \text{ in.} - xl = 3.125 \text{ in.}$$

$$a = \frac{al}{l} = \frac{3.125 \text{ in.}}{9 \text{ in.}} = 0.35$$

$$k = \frac{kl}{l} = \frac{3.5 \text{ in.}}{9 \text{ in.}} = 0.39$$

Interpolation gives C = 1.31

$$\therefore \phi R_n = P_u = C_1 C D l = (1)(1.31)(4)(9) = 47.2^k$$

12. Girder Web Strength at Weld

Yield strength per inch of girder = $\phi(0.6 F_y t_w)$

$$= 0.9(0.6)(36)(0.415) = 8.07^{k/in.}$$

Strength per inch of weld = $1.392(4) = 5.57^{k/in.}$

$5.57 < 8.07$ $\therefore$ web is adequate for full weld

$$\frac{8.07}{5.57}(47.2) = 68.4^k$$

$\therefore$ Weld Rupture controls the connection design $\Rightarrow$ $\phi V_n = 47.2^k$

Example 2. Refer to connection beam B4B framing into girder B4. (Fig. 3.1, 3.3)

Given: A W10x22 beam framing into a W18x60 Girder.
Single angle connection.
L4x3 1/2x3/8. Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.16.

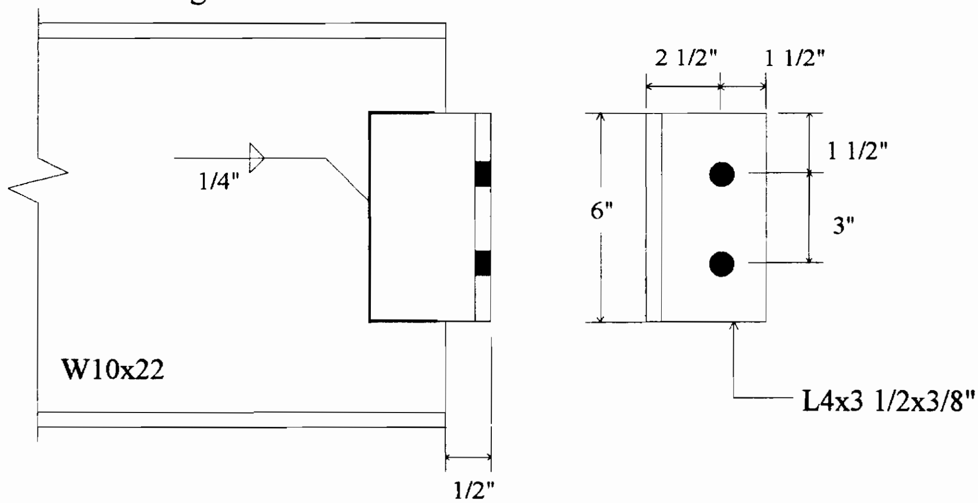


Figure 3.16

Show: All limit states that apply.

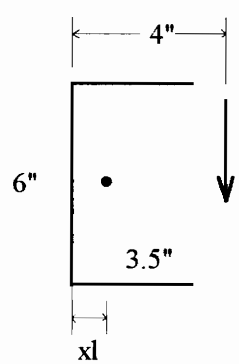
Limit States:

1. Beam Gross Shear

$$\begin{aligned}\phi V_n &= \phi(0.6\, F_y\, A_w) \\ &= 0.9(0.6)(36)(10.17 \times 0.24) \\ &= 47.4^k\end{aligned}$$

2. Weld Shear Strength (t_{weld} = 1/4 in)

Using LRFD Table XXII and the following values:



$$xl = \frac{2[3.5(1.75)]}{6 + 2(3.5)} = 0.94 \text{ in.}$$

$$al = 4 \text{ in.} - xl = 3.06 \text{ in.}$$

$$a = \frac{al}{l} = \frac{3.06 \text{ in.}}{6 \text{ in.}} = 0.51$$

$$k = \frac{kl}{l} = \frac{3.5 \text{ in.}}{6 \text{ in.}} = 0.58$$

Interpolation gives C = 1.871

$\therefore\ \phi R_n = P_u = C_1 C D l = (1)(1.871)(4)(6) = 44.9^k$

3. Beam Web Strength at Weld

Yield strength per inch of beam $= \phi(0.6F_y t_w)$

$= 0.9(0.6)(36)(0.24) = 4.67^{k/in.}$

Strength per inch of weld $= 1.392(4) = 5.57^{k/in.}$

$4.67 < 5.57$ $\therefore$ web is not adequate for full weld

$$\frac{4.67}{5.57}(44.9) = 37.7^k$$

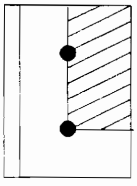
4. Angle Gross Shear

$\phi V_n = \phi(0.6F_y A_g) = 0.9(0.6)(36)(6 \times 0.375) = 43.7^k$

5. Angle Net Shear

$\phi V_n = \phi(0.6F_u A_{ns}) = 0.75(0.6)(58)(6 - 2(7/8 + 1/8)(3/8) = 39.2^k$

6. Angle Block Shear



Shear Rupture:
$0.6F_u A_{ns} = 0.6(58)[4.5 - 1.5(7/8 + 1/8)](0.375) = 39.2^k$

Tension Rupture:
$F_u A_{nt} = 58(1.5 - 0.5(7/8 + 1/8))(0.375) = 21.8^k$

Shear Rupture > Tension Rupture
$\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(1.5 \times 3/8) = 20.3^k$

$\therefore\ \phi V_n = 0.75(39.2 + 20.3) = 44.6^k$

7. Flexural Yield of Angle:

$$\phi V_n = 0.9F_y S_g / e, \qquad S_g = \frac{l^2 t}{6} = \frac{(6)^2(3/8)}{6} = 2.25\ \text{in}^2$$

$$\therefore\ \phi V_n = \frac{0.9(36)(2.25)}{\left(2 + \frac{0.24}{2}\right)} = 34.4^k$$

8. Flexural Rupture of Angle:

$$\phi V_n = 0.75F_y S_{net} / e$$

$$S_{net} = S_g - \frac{s^2 N_b \left(N_b^{\,2} - 1\right)\left[t\left(d_b + 1/8\right)\right]}{6l}$$

$$= 2.25 - \frac{(3)^2(2)\left(2^2 - 1\right)\left[0.375\left(7/8 + 1/8\right)\right]}{6(6)} = 1.69 \text{ in}^3$$

$$\therefore\ \phi V_n = \frac{0.75(58)(1.69)}{\left(2 + \frac{0.24}{2}\right)} = 34.7^k$$

9. Bolt Shear

2 7/8 in. dia. A325-N bolts, Must consider eccentricity.

Using LRFD Table X and the following values:

$b = 3$ in.

$x_o = 2$ in. $\Rightarrow$ Interpolation gives, $C = 1.18$

$n = 2$

$$\therefore\ \phi R_b = \phi r_v A_b C = 0.75(48)(0.6013)(1.18) = 25.5^k$$

10. Angle Bearing/Tear-out

edge $> 1.5d_b$, spacing $> 3d_b$ $\therefore$ Only bearing applies

Use $C = 1.18$

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = (1.18)[(0.75)(2.4)(58)(7/8)(3/8)] = 40.4^k$$

11. Girder Web Bearing/Tear-out

No edge distance concern, spacing $> 3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = (2)[(0.75)(2.4)(58)(7/8)(.415)] = 75.8^k$$

$\therefore$ Bolt shear controls the connection design $\Rightarrow$ $\phi V_n = 25.5$

3.2.5 Tee Framing Connection

Example 1. Refer to connection beam B1A framing into girder B1. (Fig. 3.1, 3.6)

Given: A W14x34 beam framing into a W18x50 girder.
Tee connection.
WT5x15. Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.17 below.

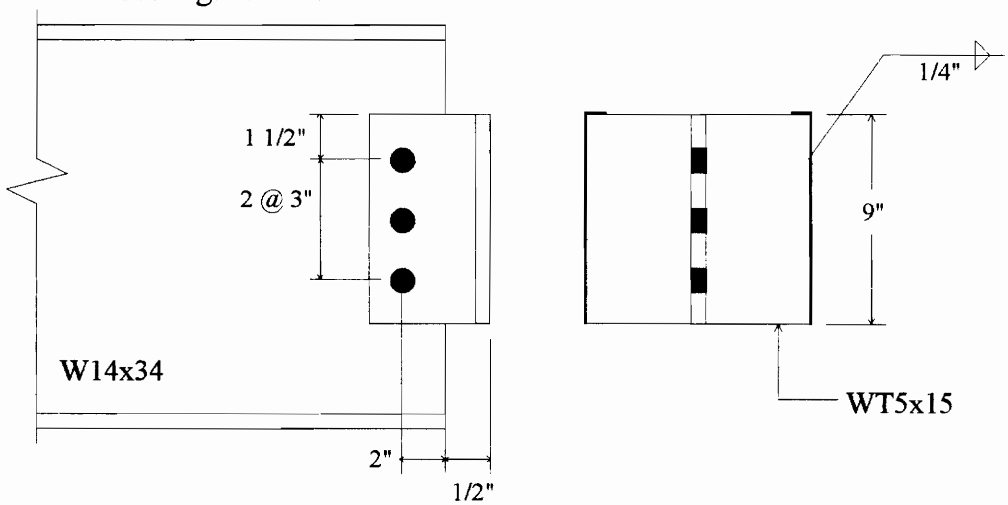


Figure 3.17

Show: All limit states that apply.

Limit States:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6\ F_y\ A_w)$$
$$= 0.9(0.6)(36)(13.98 \times 0.285)$$
$$= 77.5^k$$

2. Beam Net Shear

$$\phi V_n = \phi(0.6 F_u A_n) = 0.75(0.6)(58)[13.98 - 3(7/8 + 1/8)](0.285) = 81.7^k$$

3. Beam Web Bearing/Tear-out
No edge distance concern, s = 3 in. > $3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 0.75(3)(2.4)(58)(7/8)(.285) = 78.1^k$$

4. Bolt Shear

3 7/8 in. dia. A325-N bolts, eccentricity is ignored.

$$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(3) = 64.9^k$$

5. Tee Gross Shear

$$\phi V_n = \phi(0.6F_y A_g) = 0.9(0.6)(36)(9 \text{ x } 0.30) = 52.5^k$$

6. Tee Net Shear

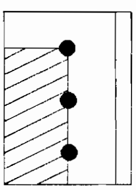
$$\phi V_n = \phi(0.6F_u A_{ns}) = 0.75(0.6)(58)(9 - 3(7/8 + 1/8)(0.30) = 47^k$$

7. Tee Bearing/Tear-out

edge $> 1.5d_b$, spacing $> 3d_b$ $\quad\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = (3)[(0.75)(2.4)(58)(7/8)(0.30)] = 77.5^k$$

8. Tee Block Shear



Shear Rupture:

$$0.6F_u A_{ns} = 0.6(58)[7.5 - 2.5(7/8 + 1/8)](0.30) = 52.2^k$$

Tension Rupture:

$$F_u A_{nt} = 58(2.735 - 0.5(7/8 + 1/8))(0.30) = 38.9^k$$

Shear Rupture > Tension Rupture

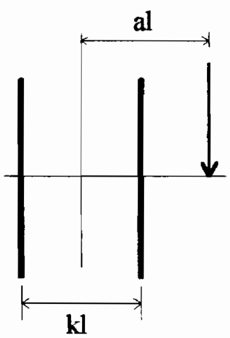
$\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(2.735 \text{ x } 0.30) = 29.5^k$

$$\therefore \ \phi V_n = 0.75(52.2 + 29.5) = 61.3^k$$

9. Weld Shear Strength ($t_{weld} = 1/4$ in)

Using LRFD Table XXIII (special case) and the following values:



Ignoring the returns,

al = 2.5 in., kl = 5.77 in.

$$a = \frac{al}{l} = \frac{2.5 \text{ in.}}{9 \text{ in.}} = 0.278$$

$$k = \frac{kl}{l} = \frac{5.77 \text{ in.}}{9 \text{ in.}} = 0.641$$

Interpolation gives C = 2.157

$\therefore\ \phi R_n = P_u = C_1 C D l = (1)(2.157)(4)(9) = 77.7^k$

10. Girder Web Strength at Weld

$$\phi R_n = \phi(0.6F_y)L_{weld}t_w = 0.9(0.6)(36)(9)(0.355)\text{x}2 = 124.2^k$$

$\therefore$ Tee Stem Net Shear controls the connection design $\Rightarrow\ \phi V_n = 47^k$

Note: The following are additional requirements to ensure ductility (Murray 1993).

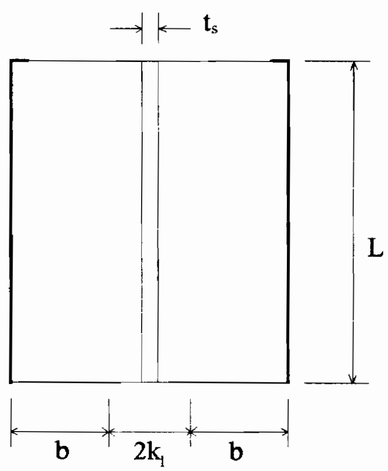
$$w_{min} = 0.0158\frac{F_y t_f}{b}\left(\frac{b^2}{L^2}+2\right) \le \frac{3}{4}t_s$$

$$d_{b\,min} = 0.162 t_f\sqrt{\frac{F_y}{b}\left(\frac{b^2}{L^2}+2\right)} \le 0.69\sqrt{t_s}$$

also,

$$t_s \le \left(\frac{d_b}{2}\right)+\frac{1}{16}$$

$$t_s \ge \frac{L}{64}$$



3.3 Moment Connection Design Examples

3.3.1 Flange Welded Moment Connection

Example 1. Refer to connection beam B1 framing into column C1. (Fig. 3.1, 3.5)

Given: A W18x50 beam framing into a W12x170 column.
Flange welded, web bolted moment connection.
PL 1/4x4 1/2x9
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.18 below.

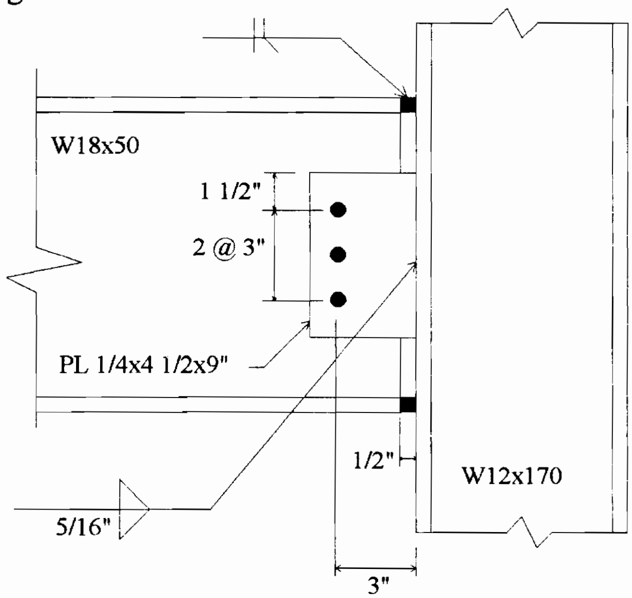


Figure 3.18

Show: All limit states that apply.

Limit States:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6F_yA_w) = 0.9(0.6)(36)(17.99 \times 0.355) = 124.2^k$$

2. Beam Net Shear

$$\phi V_n = \phi(0.6F_uA_n) = 0.75(0.6)(58)[17.99 - 3(7/8 + 1/8)](0.355) = 138.9^k$$

3. Beam Web Bearing/Tear-out
Eccentricity is ignored
No edge distance concern & s = 3 in. > $3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 3 \times 0.75(2.4)(58)(7/8)(.355) = 97.3^k$$

4. <u>Bolt Shear</u>
   3 7/8 in. dia. A325-N bolts

$$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(3) = 64.9^k$$

5. <u>Plate Gross Shear</u>

$$\phi V_n = \phi(0.6F_y A_g) = 0.9(0.6)(36)(9 \times .25) = 43.7^k$$

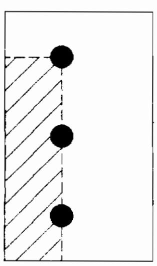
6. <u>Plate Net Shear</u>

$$\phi V_n = \phi(0.6F_u A_{ns}) = 0.75(0.6)(58)[9 - 3(7/8 + 1/8)]0.25 = 39.2^k$$

7. <u>Plate Bearing / Tear-out</u>
   edge = 1.25 in. > $1.5d_b$ & spacing = 3 > $3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 3[0.75(2.4)(58)(7/8)(.25)] = 68.5^k$$

8. <u>Plate Block Shear</u>



Shear Rupture:

$$0.6F_u A_{ns} = 0.6(58)(7.5 - 2.5(7/8 + 1/8))0.25 = 43.5^k$$

Tension Rupture:

$$F_u A_{nt} = 58(1.5 - 0.5(7/8 + 1/8))0.25 = 14.5^k$$

Shear Rupture > Tension Rupture
$\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(1.5 \times 0.25) = 13.5^k$

$$\therefore \phi V_n = 0.75(43.5 + 13.5) = 42.8^k$$

9. <u>Weld Rupture</u>

$$\phi R_n = 1.392(5)(9)(2) = 125.3^k$$

10. <u>Column Flange Shear Yield at Weld</u>

$$\phi R_n = 0.9(0.6)(36)(9 \times 1.56)(2) = 545.9^k$$

11. Weld at Beam and Column Flange

Full Penetration groove weld, $F_u = 70$ ksi

Be sure weld is compatible with both materials

Check for stiffener requirement:

If $\phi R_n \le$ Flange force $= F_{fu} = M_u / (d - t_f)$, then stiffeners are required.

12. Column Flange Bending

$$\phi R_n = \phi 6.25 t_f^2 F_{yf} = 0.9(6.25)(1.56)^2(36) = 492.8^k$$

13. Column Web Yielding

$$\phi R_n = \phi(t_f + 5k)F_{yw}t_w = 1.0[0.57 + (5 \times 2.25)](36)(0.96) = 408.5^k$$

14. Column Web Crippling

$$\phi R_n = \phi 135 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw}t_f}{t_w}}$$

$$\phi R_n = (0.75)135(0.96)^2\left[1 + 3\left(\frac{0.57}{14.03}\right)\left(\frac{0.96}{1.56}\right)^{1.5}\right]\sqrt{\frac{36(1.56)}{0.96}}$$

$$\phi R_n = 756^k$$

15. Column Web Buckling

$$\phi R_n = \phi\frac{4100 t_{wc}^3\sqrt{F_{yc}}}{d_c} = (0.75)\frac{4100(0.96)^3\sqrt{36}}{9.5} = 2062^k$$

Assuming $F_{fu} << \phi R_n$, no stiffeners are required.

$\therefore$ Plate Block Shear controls the connection design $\Rightarrow$ $\phi R_n = 42.8^k$

3.3.2 Flange Plate Bolted Moment Connection

Example 2. Refer to connection beam B2 framing into column C1. (Fig. 3.1, 3.3, 3.5)

Given: A W18x60 beam framing into a W12x170 column (weak axis).
Flange plate bolted, web bolted moment connection.
PL 1/2x10x1'-6 1/2"
2-PL 3/4x10 7/8x1'- 3 1/2"
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.19 below.

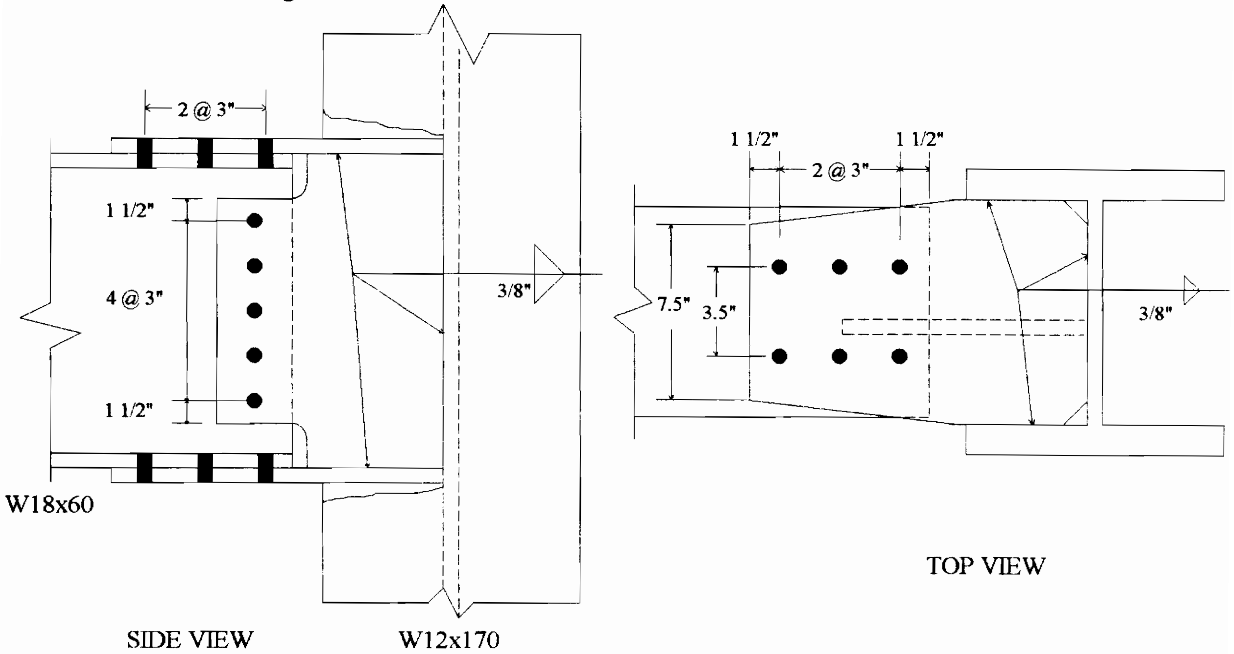


Figure 3.19

Show: All limit states that apply.

Limit States:

Shear:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6F_yA_w) = 0.9(0.6)(36)(18.24 \times 0.415) = 147.2^k$$

2. Beam Net Shear

$$\phi V_n = \phi(0.6F_uA_n) = 0.75(0.6)(58)[18.24 - 5(7/8 + 1/8)](0.415) = 143.4^k$$

3. <u>Beam Web Bearing/Tear-out</u>
   Eccentricity is ignored
   No edge distance concern & s = 3 in. > $3d_b$ $\quad\therefore$ Only bearing applies

   $$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 5 \times 0.75(2.4)(58)(7/8)(.415) = 189.6^k$$

4. <u>Bolt Shear</u>
   5 7/8 in. dia. A325-N bolts

   $$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(5) = 108.2^k$$

5. <u>Plate Gross Shear</u>

   $$\phi V_n = \phi(0.6F_y A_g) = 0.9(0.6)(36)(15 \times 0.5) = 145.8^k$$

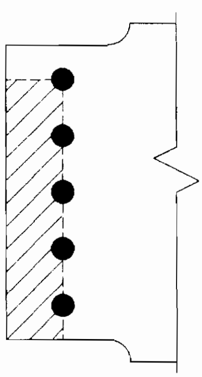
6. <u>Plate Net Shear</u>

   $$\phi V_n = \phi(0.6F_u A_{ns}) = 0.75(0.6)(58)[15 - 5(7/8 + 1/8)]0.5 = 130.5^k$$

7. <u>Plate Bearing / Tear-out</u>
   edge = 1.5 in. > $1.5d_b$ & spacing = 3 > $3d_b$ $\quad\therefore$ Only bearing applies

   $$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 5[0.75(2.4)(58)(7/8)(0.5)] = 228.4^k$$

8. <u>Plate Block Shear</u>



Shear Rupture:

$$0.6F_u A_{ns} = 0.6(58)[13.5 - 4.5(7/8 + 1/8)]0.5 = 156.5^k$$

Tension Rupture:

$$F_u A_{nt} = 58[1.5 - 0.5(7/8 + 1/8)]0.5 = 29^k$$

Shear Rupture > Tension Rupture
$\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(1.5 \times 0.5) = 27^k$

$$\therefore \phi V_n = 0.75(156.5 + 27) = 137.7^k$$

9. Plate Yield at Weld (at column web)

$$\phi V_n = 0.9(0.6 \times 36)[18.5 - 2(0.75)](0.5 \times 2) = 330.5^k$$

10. Weld Rupture (Shear only)

$$\phi R_n = 1.392(6)[18.5 - 2(0.75)](2) = 284^k$$

11. Column Web Shear Yield at Weld

$$\phi R_n = 0.9(0.6)(36)[18.5 - 2(0.75)](0.96)x2 = 634.5^k$$

Moment: Compare force limit states with Flange force $= F_{fu} = M_u / (d - t_f)$

12. Web Plate Yield at Weld (at flange plates)

$$\phi V_n = 0.9(0.6 \times 36)[6 - 0.75](0.5 \times 2) = 102.1^k$$

13. Flange Plate Yield at Weld

$$\phi V_n = 0.9(0.6 \times 36)[6 - 0.75](0.75 \times 2) = 153.1^k$$

14. Beam Flexural Strength Reduction

$$0.75F_uA_{fn} = 0.75(58)[7.55 - 2(7/8 + 1/8)](0.695) = 167.8^k$$
$$0.9F_yA_g = 0.9(36)[7.55](0.695) = 170^k$$

$0.9F_yA_g \geq 0.75F_uA_{fn}$ $\quad\therefore$ Reduced capacity due to flange holes

$$A_{fe} = \frac{0.75F_u}{0.9F_y}A_{fn} = \frac{0.75(58)}{0.9(36)}[7.55 - 2(1)](0.695) = 5.18 \text{ in}^2$$

$$A_{removed} = A_{fg} - A_{fe} = 7.55(0.695) - 5.18 = 0.08 \text{ in}^2$$

$$M_p = 369 \text{ k-ft}$$

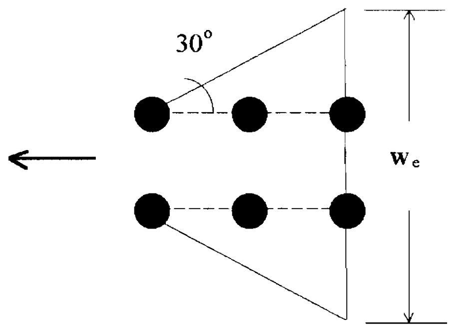
$$M_{p\,reduced} = 369 - 36(0.08)(18.24 - 0.695)/12 = 364.8 \text{ k-ft}$$

$$\therefore \phi M_p = 328.3 \text{ k-ft}$$

15. Flange Plate Tension Yield

$$\phi T_n = \phi F_y A_g \quad A_g = \min \left| \begin{array}{l} w_e\, t_p \\ w_p\, t_p \end{array} \right.$$

w_e using the Whitmore Model below (Whitmore 1952).



$w_e = 3.5 + 2[(3 + 3)\tan 30°] = 10.43$ in.

$w_p = 6.14$ in. (at last bolt group)

$\therefore\ \phi T_n = 0.9(36)(6.14 \times 0.75) = 149.2^k$

16. Flange Plate Tension Rupture

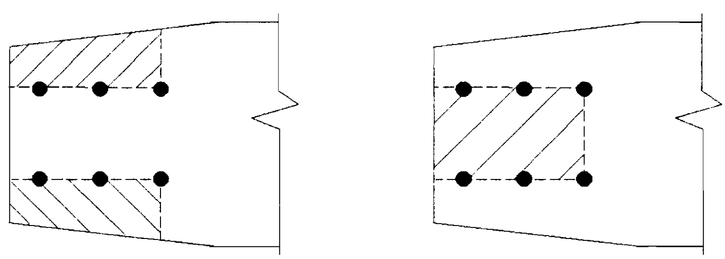
$\phi T_n = \phi F_u A_n = (0.75)(58)[6.14 - 2(7/8 + 1/8)](0.75) = 135.1^k$

17. Tension Flange Plate Bearing/Tear-out

edge = 1.5 in. > $1.5d_b$ & spacing = 3 > $3d_b$ $\quad\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 6[0.75(2.4)(58)(7/8)(0.75)] = 411.1^k$

18. Tension Flange Plate Block Shear



Case 1 Case 2

By inspection Case 2 controls.

Shear Rupture:

$0.6F_u A_{ns} = 0.6(58)(7.5 - 2.5(7/8 + 1/8))0.75\text{x}2$
$= 261^k$

Tension Rupture:

$F_u A_{nt} = 58(3.5 - 1(7/8 + 1/8))0.75$
$= 108.8^k$

Shear Rupture > Tension Rupture $\quad\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(3.5 \times 0.75) = 94.5^k$

$\therefore \ \phi V_n = 0.75(261 + 94.5) = 266.6^k$

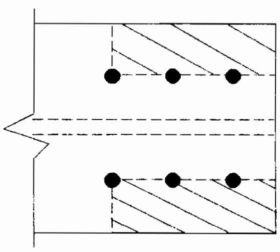
19. Bolt Shear
 6 7/8 in. dia. A325-N bolts

$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(6) = 129.9^k$

20. Beam Flange Bearing / Tear-out
 edge = 1.5 in. > $1.5d_b$ & spacing = 3 > $3d_b$ $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 6[0.75(2.4)(58)(7/8)(0.695)] = 381^k$

21. Beam Flange Block Shear



Shear Rupture:

$0.6F_u A_{ns} = 0.6(58)(7.5 - 2.5(7/8 + 1/8))0.695\text{x}2$
$= 241.9^k$

Tension Rupture:

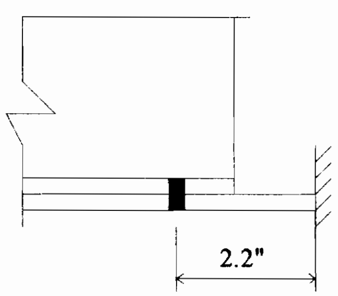
$F_u A_{nt} = 58(2.025 - 0.5(7/8 + 1/8))0.695\text{x}2$
$= 122.9^k$

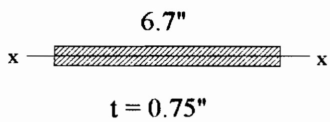
Shear Rupture > Tension Rupture $\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(2.025 \times 0.695)\text{x}2 = 101.3^k$

$\therefore \ \phi V_n = 0.75(241.9 + 101.3) = 257.4^k$

22. Compression Flange Plate Capacity





$I_x = 1/12(6.7)(0.75)^3 = 0.236 \text{ in}^4$

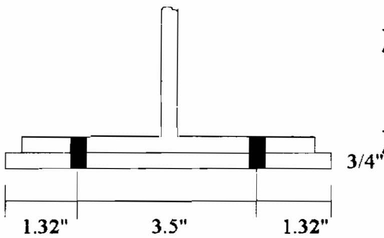
$A_g = 6.7(0.75) = 5.025 \text{ in}^2$

$r_x = \sqrt{\dfrac{I_x}{A_g}} = \sqrt{\dfrac{0.236}{5.025}} = 0.22 \text{ in}$

use k = 0.8 $\quad kL/r = 0.8(2.2)/0.22 = 8.0 \Rightarrow \quad \phi F_{cr} = 30.5$ ksi

$\therefore\ \phi R_n = 30.5(5.025) = 153.3^k$

23. Compression Flange Local Buckling



$$\lambda = \frac{1.32}{0.75} = 1.76 < \lambda_r = \frac{95}{\sqrt{F_y}} \quad \therefore \text{ ok}$$

$$\lambda = \frac{3.5}{0.75} = 4.67 < \lambda_r = \frac{253}{\sqrt{F_y}} \quad \therefore \text{ ok}$$

24. Weld at Column Flanges

$\phi R_n = 2(2)(1.392)(6)(5.8 - 3/8 - 3/4) = 156.2^k$

25. Column Flange Yield at Weld

$\phi R_n = 0.9(0.6)(36)(5.8 - 3/8 - 3/4)(1.56) = 141.8^k$

26. Weld at Column Web

$\phi R_n = 2(1.392)(6)[10.91 - 2(3/4)] = 157.2^k$

27. Column Web Yield at Weld

$\phi R_n = 0.9(0.6)(36)[10.91 - 2(3/4)](0.96) = 292.7^k$

$\therefore$ Bolt Shear controls the connection design (for shear portion) $\Rightarrow \ \phi R_n = 108.2^k$

Web Plate Yield at Flange Plate Weld controls the connection design (for moment portion) $\Rightarrow \ \phi R_n = 102.1^k$

3.3.3 Flange Plate Welded Moment Connection

Example 3. Refer to connection beam B4 framing into column C1. (Fig. 3.1, 3.4, 3.6)

Given: A W18x60 beam framing into a W12x170 column (weak axis).
Flange welded, web bolted moment connection.
PL 1/2x10x1'-4 3/4"
2-PL 3/4x10 7/8x1'-6 1/2"
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.20 below.

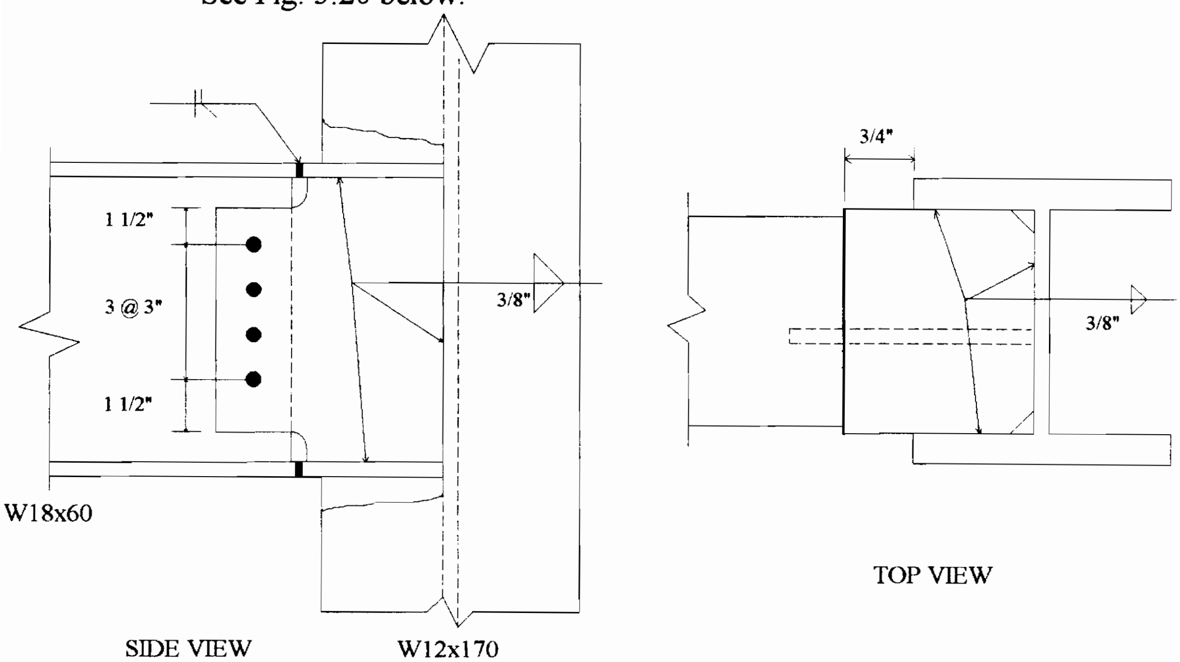


Figure 3.20

Show: All limit states that apply.

Limit States:

Shear:

1. Beam Gross Shear

$$\phi V_n = \phi(0.6F_yA_w) = 0.9(0.6)(36)(18.24 \times 0.415) = 147.2^k$$

2. Beam Net Shear

$$\phi V_n = \phi(0.6F_uA_n) = 0.75(0.6)(58)[18.24 - 4(7/8 + 1/8)](0.415) = 154.2^k$$

3. Beam Web Bearing/Tear-out
Eccentricity is ignored
No edge distance concern & s = 3 in. > $3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 4 \times 0.75(2.4)(58)(7/8)(.415) = 151.6^k$$

4. Bolt Shear
4 7/8 in. dia. A325-N bolts

$$\phi R_b = \phi r_v A_b N_b = 0.75(48)(0.6013)(4) = 86.6^k$$

5. Plate Gross Shear

$$\phi V_n = \phi(0.6 F_y A_g) = 0.9(0.6)(36)(12 \times 0.5) = 116.6^k$$

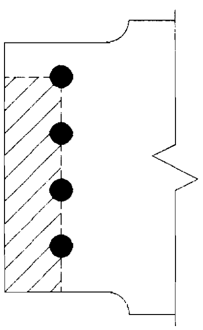
6. Plate Net Shear

$$\phi V_n = \phi(0.6 F_u A_{ns}) = 0.75(0.6)(58)[12 - 4(7/8 + 1/8)]0.5 = 104.4^k$$

7. Plate Bearing / Tear-out
edge = 1.5 in. > $1.5d_b$ & spacing = 3 > $3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 4[0.75(2.4)(58)(7/8)(0.5)] = 182.7^k$$

8. Plate Block Shear



Shear Rupture:

$$0.6 F_u A_{ns} = 0.6(58)[10.5 - 3.5(7/8 + 1/8)]0.5 = 121.8^k$$

Tension Rupture:

$$F_u A_{nt} = 58[1.5 - 0.5(7/8 + 1/8)]0.5 = 29^k$$

Shear Rupture > Tension Rupture
$\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(1.5 \times 0.5) = 27^k$

$$\therefore \phi V_n = 0.75(121.8 + 27) = 111.6^k$$

9. Plate Yield at Weld (at column web)

$\phi V_n = 0.9(0.6 \times 36)[16.75 - 2(0.75)](0.5 \times 2) = 296.4^k$

10. Weld Rupture (Shear only)

$\phi R_n = 1.392(6)[16.75 - 2(0.75)](2) = 254.7^k$

11. Column Web Shear Yield at Weld

$\phi R_n = 0.9(0.6)(36)[16.75 - 2(0.75)](0.96) = 569.2^k$

Moment: Compare force limit states with Flange force $= F_{fu} = M_u / (d - t_f)$

12. Web Plate Yield at Weld (at flange plates)

$\phi V_n = 0.9(0.6 \times 36)[6 - 0.75](0.5 \times 2) = 102.1^k$

13. Flange Plate Yield at Weld (at web plate)

$\phi V_n = 0.9(0.6 \times 36)[6 - 0.75](0.75 \times 2) = 153.1^k$

14. Flange Plate TensionYield at Weld

$\phi R_n = 0.9(36)[7.55 \times 0.75] = 183.5^k$

15. Beam Flange Tension Yield at Weld

$\phi R_n = 0.9(36)[7.55 \times 0.695] = 170^k$

16. Weld Rupture

Full Penetration Groove Weld, $F_u = 70$ ksi

Be sure weld matches base material.

17. Flange Plate Tension Yield

$\phi R_n = 0.9(36)[10.91 \times 0.75] = 265.1^k$

18. Flange Plate Tension Rupture

$\phi R_n = 0.75(58)[10.91 \times 0.75] = 356^k$

19. Flange Plate Weld Rupture at Column Web

$\phi R_n = 2\text{x}1.392(6)[10.91 - 2(0.75)] = 157.2^k$

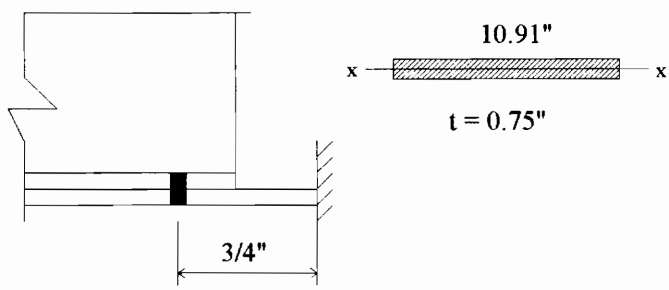
20. Flange Plate Weld Rupture at Column Flanges

$\phi V_n = 2\text{x}2\text{x}1.392(6)[5.8 - 3/8 - 0.75] = 156.2^k$

21. Flange Plate Shear Yield at Weld at Column Flanges

$\phi V_n = 0.9(0.6 \times 36)[5.8 - 3/8 - 0.75](0.75)\text{x}2\text{x}2 = 272.6^k$

22. Compression Flange Plate Capacity



$I_x = 1/12(10.91)(0.75)^3 = 0.384 \text{ in}^4$

$A_g = 10.91(0.75) = 8.18 \text{ in}^2$

$$r_x = \sqrt{\frac{I_x}{A_g}} = \sqrt{\frac{0.384}{8.18}} = 0.217 \text{ in}$$

use $k = 0.8$ $\quad kL/r = 0.8(.75)/0.217 = 2.77 \quad \Rightarrow \quad \phi F_{cr} = 30.59$ ksi

$\therefore \ \phi R_n = 30.59(8.18) = 250.2^k$

23. Compression Flange Local Buckling

$$\lambda = \frac{10.91}{0.75} = 14.5 < \lambda_r = \frac{253}{\sqrt{F_y}} \quad \therefore \text{ ok}$$

$\therefore$ Bolt Shear controls the connection design (for shear portion) $\Rightarrow \ \phi R_n = 86.6^k$

Web Plate Yield at Flange Plate Weld controls the connection design (for moment portion) $\Rightarrow \ \phi R_n = 102.1^k$

3.3.4 Extended End-Plate Moment Connection

Example 3. Refer to connection beam B4 framing into column C1. (Fig. 3.1, 3.4, 3.5)

Given: A W18x60 beam framing into a W12x170 column (weak axis).
4-bolt extended, unstiffened moment end-plate connection.
PL 1x7 1/2x2'- 3 1/4"
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.21 below.

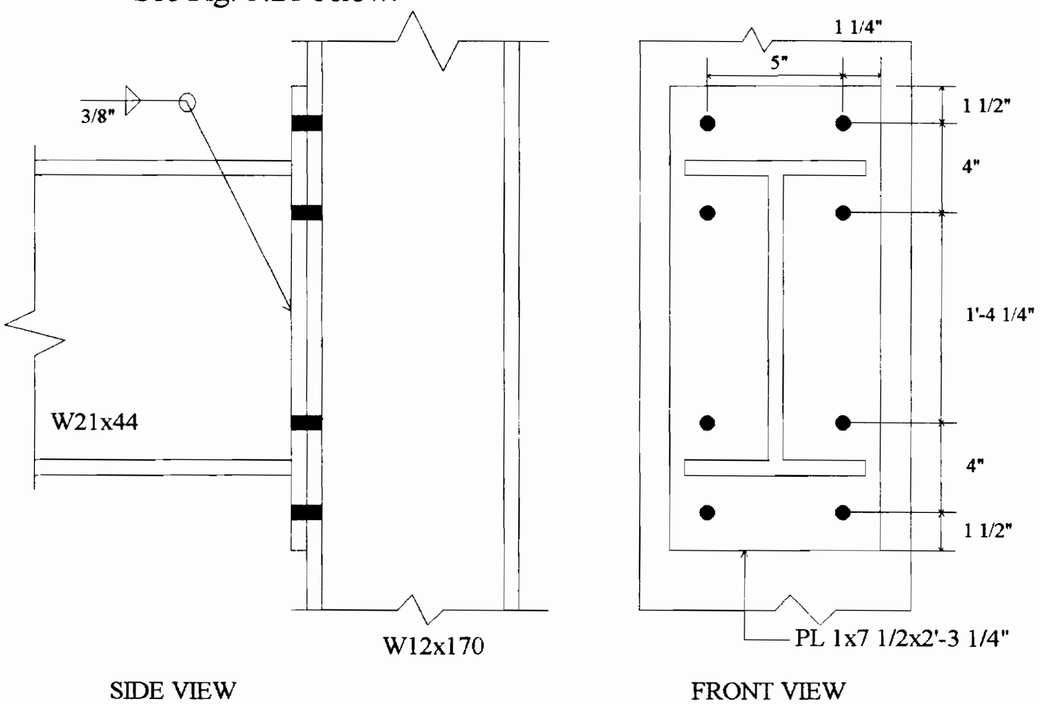


Figure 3.21

Show: All limit states that apply.

Note: Refer to *Extended End-Plate Moment Connections* (Murray 1990).

Limit States:

1. Beam Gross Shear

$\phi V_n = \phi(0.6F_yA_w) = 0.9(0.6)(36)(20.66 \times 0.35) = 140.6^k$

2. Bolt Shear and Tension

- Tension: $\phi R_b = 0.75(90)(0.6013) = 40^{k/bolt} = 160^k$
- Shear: $\phi R_b = 0.75(48)(0.6013) = 21.6^{k/bolt} = 86.6^k$

3. <u>End-Plate Yield</u>

$b_p \leq b_{fb} + 1 = 7.5$ in. $\therefore$ ok
$g \leq b_{fb} = 6.5$ in. $\therefore$ ok

Effective bolt distance,

$$p_e = p_f - d_b/2 - 0.707w = 1.30$$
$$\frac{A_f}{A_w} = \frac{6.5(0.45)}{20.66(0.35)} = 0.405$$

$$p_e / d_b = 1.30 / 0.875 = 1.49$$
$$C_b = \left(\frac{b_f}{b_p}\right)^3 = \left(\frac{6.5}{7.5}\right)^3 = 0.931$$

$C_a = 1.36$ (from AISC moment end-plate design guide)
$$\alpha_m = C_a C_b \left(\frac{A_f}{A_w}\right)^{1/3} \left(\frac{p_e}{d_b}\right)^{1/4}$$
$$= 1.36(0.931)(0.405)^{1/3}(1.49)^{1/4} = 1.04$$
$$M_{eu} = \frac{\alpha_m F_{fu} p_e}{4}, \text{ let } F_{fu} = 4(40) = 160^k$$
$$M_{eu} = \frac{1.04(160)(1.3)}{4} = 54.1^{k\text{-}ft}$$
$$\therefore t_{p\,req'd} = \sqrt{\frac{4M_{eu}}{0.9F_y b_p}} = 0.94 \text{ in.} < 1 \text{ in.}, \text{ ok}$$

4. <u>Weld at Beam Web to End-Plate</u>

Note: Weld between end-plate and beam need to meet the requirement for tension in the web near the tension bolts and the requirement for beam shear.

Req'd weld to develop web tension,
$$D_{req'd} = \frac{\phi F_y t_w}{2(1.392)} = \frac{0.9(36)(0.35)}{2(1.392)} = 4.07 < 6 \therefore \text{ok}$$

Req'd weld for shear, (assume $V_u = 10^k$)
$$D_{req'd} = \frac{V_u}{2(1.392)\left(\frac{d}{2} - t_f\right)} = \frac{10}{2(1.392)\left(\frac{d}{2} - t_f\right)} << 6 \therefore \text{ok}$$

Column Side Limit States:

5. Column Web Yield

$$\phi R_n = \phi F_{yc} t_{wc}(t_{fb} + 2w + t_p + 6k)$$
$$= 1.0(36)(0.96)[0.45 + 2(0.375) + 6(2.25)] = 542.6^k$$

6. Column Web Crippling

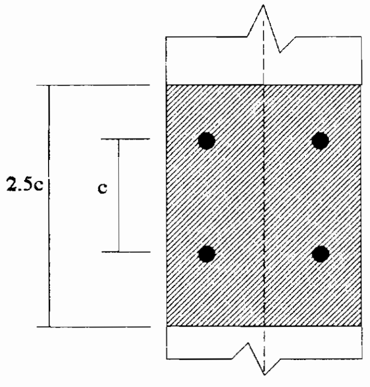
Will not govern

7. Column Web Buckling

Will not govern

8. Column Flange Bending

Analyze column as an end-plate with,



$t_p = t_{fc}$
$b_p = 2.5(p_f + t_{fb} + p_f)$, $c = (p_f + t_{fb} + p_f)$
$b_f / b_p = 1.0$
$A_f / A_w = 1.0$

$b_p = 2.5(4) = 10$

$p_e = g/2 - d_b/2 - k_1 = 5/2 - .875/2 - 1.125 = 1.16$ in.

$$\alpha_m = C_a C_b \left(\frac{A_f}{A_w}\right)^{1/3} \left(\frac{p_e}{d_b}\right)^{1/4}$$

$$= 1.36(1.0)(1.0)^{1/3}\left(\frac{1.16}{.875}\right)^{1/4} = 1.46$$

$$M_{eu} = \frac{\phi F_y b_{fc} t_{fc}^2}{4} = \frac{0.9(36)(12.57)(1.56)^2}{4} = 247.8^{k\text{-in}}$$

$$F_{cap} = \text{unstiffened column capacity} = \phi Rn$$

$$= \frac{4M_{eu}}{\alpha_m p_e} = \frac{4(247.8)}{1.46(1.16)} = 585.3^k$$

If $\phi R_n < F_{fu}$, stiffeners and welds need to be designed to carry $F_{fu} - \phi R_{n\,min}$.

3.4 Miscellaneous Connection Design Examples

3.4.1 Beam and Column Splice Connections

<u>Example 1.</u> **Beam Splice** - Refer to beam B3 splice connection. (Fig. 3.5, 3.6)

<u>Given:</u> 2-W21x44 beam members in a moment splice connection.
3-Plate, bolted beam moment splice connection.
2-PL 1/2x6 1/2x1'-6 1/2" (flange plates)
2-PL 1/4x6 1/2x1'-3" (web plates)
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.22 below.

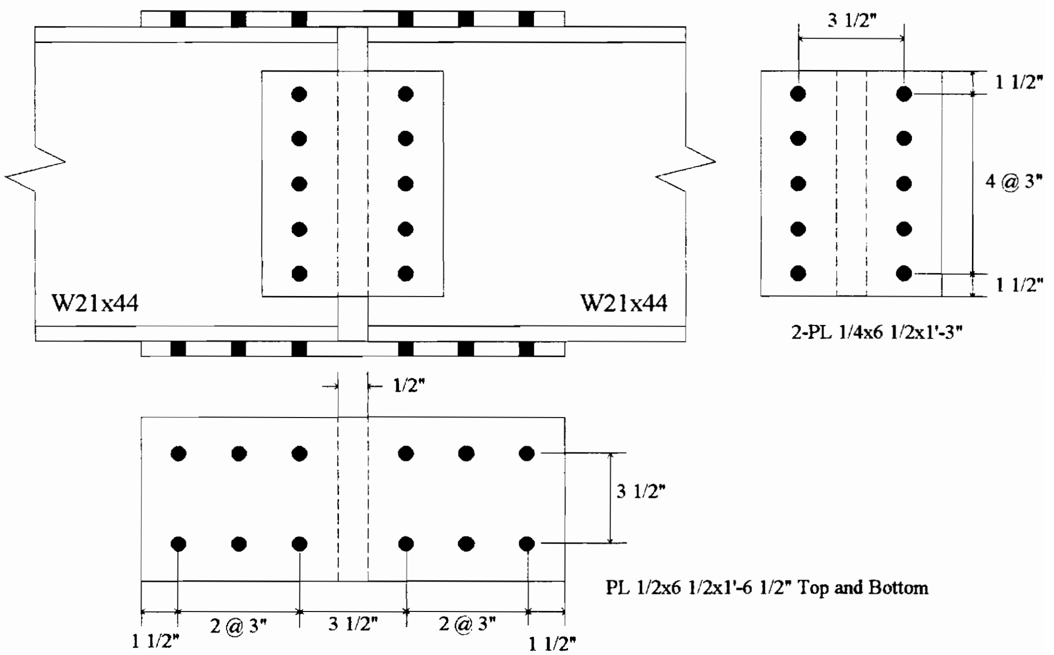


Figure 3.22

<u>Show:</u> All limit states that apply.

<u>Limit States:</u>

1. <u>Beam Gross Shear</u>

$$\phi V_n = \phi(0.6F_yA_w) = 0.9(0.6)(36)(20.66)(0.35) = 140.6^k$$

2. <u>Beam Net Shear</u>

$$\phi V_n = \phi(0.6F_uA_n) = 0.75(0.6)(58)[20.66 - 5(7/8 + 1/8)](0.35) = 143.1^k$$

3. Beam Web Bearing/Tear-out
Must consider eccentricity

Using LRFD Table X and the following values:

$b = 3$ in.
$x_o = 2$ in. $\therefore$ C = 4.40 (effective number of bolts)
$n = 5$

No edge distance concern & $s = 3$ in. $> 3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 4.40 \times 0.75(2.4)(58)(7/8)(.35) = 140.7^k$$

4. Bolt Shear
Consider eccentricity. Use effective number of bolts, C = 4.40

$$\phi R_b = \phi r_v A_b C m = 0.75(48)(0.6013)(4.40)x2 = 190.5^k$$

5. Web Plate Gross Shear

$$\phi V_n = \phi(0.6F_y A_g) = 0.9(0.6)(36)(15 \times .25) = 72.9^{k/plate} = 145.8^k$$

6. Web Plate Net Shear

$$\phi V_n = \phi(0.6F_u A_{ns}) = 0.75(0.6)(58)[15 - 5(7/8 + 1/8)](0.25)x2 = 130.5^k$$

7. Web Plate Bearing / Tear-out
edge $> 1.5d_b$ & spacing $= 3 > 3d_b$ $\therefore$ Only bearing applies

Note: Use effective number of bolts, C = 4.40

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 4.40[0.75(2.4)(58)(7/8)(.25)]x2 = 201^k$$

8. Top Flange Plate Tension Yield

$$\phi T_n = 0.9(36)(6.5 \times 0.5) = 105.3^k$$

9. Top Flange Plate Tension Rupture

$$\phi T_n = \phi F_u A_e \qquad A_e = \min \left| \begin{array}{l} 0.85A_g \\ A_g - \Sigma A_{hole} \end{array} \right. \qquad \text{where,}$$

$$A_e = \min \left| \begin{array}{l} 0.85(6.5 \times 0.5) = 2.7625 \text{ in}^2 \\ (6.5 \times 0.5) - 2(7/8 + 1/8)0.5 = 2.25 \text{ in}^2 \end{array} \right.$$

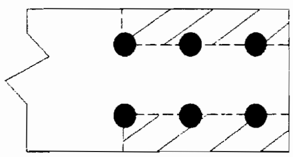
$$\therefore \phi T_n = 0.75(58)(2.25) = 97.9^k$$

10. Flange Plate Bearing / Tear-out

edge $> 1.5d_b$ & spacing $= 3 > 3d_b$ ∴ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 6[0.75(2.4)(58)(7/8)(0.5)] = 274.1^k$$

11. Flange Plate Block Shear



Controlling Case

Shear Rupture:

$$0.6F_u A_{nv} = 0.6(58)[7.5 - 2.5(7/8 + 1/8)](0.5)2 = 174^k$$

Tension Rupture:

$$F_u A_{nt} = 58[1.5 - 0.5(7/8 + 1/8)](0.5)2 = 58^k$$

Shear Rupture > Tension Rupture ∴ Shear Rupture controls

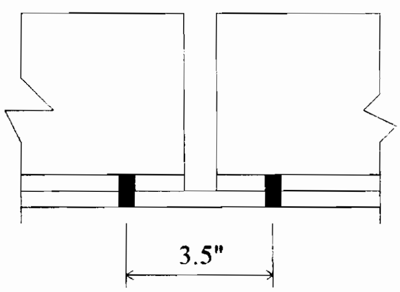
Tension Yield: $F_y A_{gt} = 36(1.5 \times 0.5)2 = 54^k$

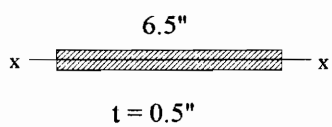
∴ $\phi T_n = 0.75(174 + 54) = 171^k$

12. Bolt Shear

$$\phi R_n = 0.75(48)(0.6013)x6 = 129.9^k$$

13. Compression Flange Plate Capacity





$$I_x = 1/12(6.5)(0.5)^3 = 0.068 \text{ in}^4$$

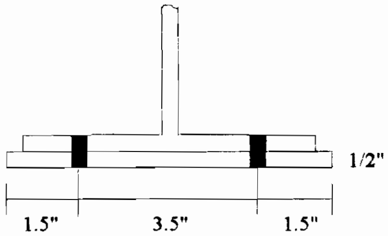
$$A_g = 6.5(0.5) = 3.25 \text{ in}^2$$

$$r_x = \sqrt{\frac{I_x}{A_g}} = \sqrt{\frac{0.068}{3.25}} = 0.145 \text{ in}$$

use $k = 0.8$ $kL/r = 0.8(3.5)/0.145 = 24.1$ ⇒ $\phi F_{cr} = 29.68$ ksi

∴ $\phi R_n = 29.68(3.25) = 96.5^k$

14. Compression Flange Local Buckling



$$\lambda = \frac{1.5}{0.5} = 3 < \lambda_r = \frac{95}{\sqrt{F_y}} \quad \therefore \text{ ok}$$

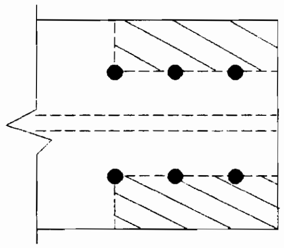
$$\lambda = \frac{3.5}{0.5} = 7 < \lambda_r = \frac{253}{\sqrt{F_y}} \quad \therefore \text{ ok}$$

15. Beam Flange Bearing / Tear-out

edge = 1.5 in. > $1.5d_b$ & spacing = 3 > $3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 6[0.75(2.4)(58)(7/8)(0..45)] = 246.6^k$$

16. Beam Flange Block Shear



Shear Rupture:

$$0.6F_u A_{ns} = 0.6(58)[(7.5 - 2.5(7/8 + 1/8)]0.45x2 = 156.6^k$$

Tension Rupture:

$$F_u A_{nt} = 58[1.5 - 0.5(7/8 + 1/8)]0.45x2 = 52.2^k$$

Shear Rupture > Tension Rupture $\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(1.5 \text{ x } 0.45)x2 = 48.6^k$

$\therefore \ \phi V_n = 0.75(156.6 + 48.6) = 153.9^k$

17. Beam Flexural Strength Reduction

$$0.75F_u A_{fn} = 0.75(58)[6.5 - 2(7/8 + 1/8)](0.45) = 88.1^k$$
$$0.9F_y A_g = 0.9(36)(0.65 \text{ x } 0.45) = 94.8^k$$

$0.9F_y A_g \geq 0.75F_u A_{fn}$ $\therefore$ Reduced capacity due to flange holes

$$A_{fe} = \frac{0.75F_u}{0.9F_y} A_{fn} = \frac{0.75(58)}{0.9(36)}[6.5 - 2(1)](0.45) = 2.72 \text{ in}$$

$$A_{removed} = A_{fg} - A_{fe} = 6.5(0.45) - 2.72 = 0.21 \text{ in}^2$$

$$M_p = 287 \text{ k-ft}$$

$$M_{p\,reduced} = 287 - 36(0.21)(20.66 - 0.45)/12 = 274.3 \text{ k-ft}$$

$$\therefore \ \phi M_p = 246.8 \text{ k-ft}$$

∴ Web Plate Shear Rupture controls the connection design
(for shear portion) ⇒ $\phi R_n = 130.5^k$

Flange Plate Compressive Capacity controls the connection design
(for moment portion) ⇒ $\phi R_n = 96.5^k$

Example 2. **Column Splice** - Refer to C1 to C2 column splice connection. (Fig. 3.3, 3.5)

Given: A W12x170 and W12x106 column in a connection.
2-Plate, bolted/welded column splice connection.
PL 5/8x9x9" (web plate)
PL 1x8 1/2x1'-9" (flange plate)
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.23 below.

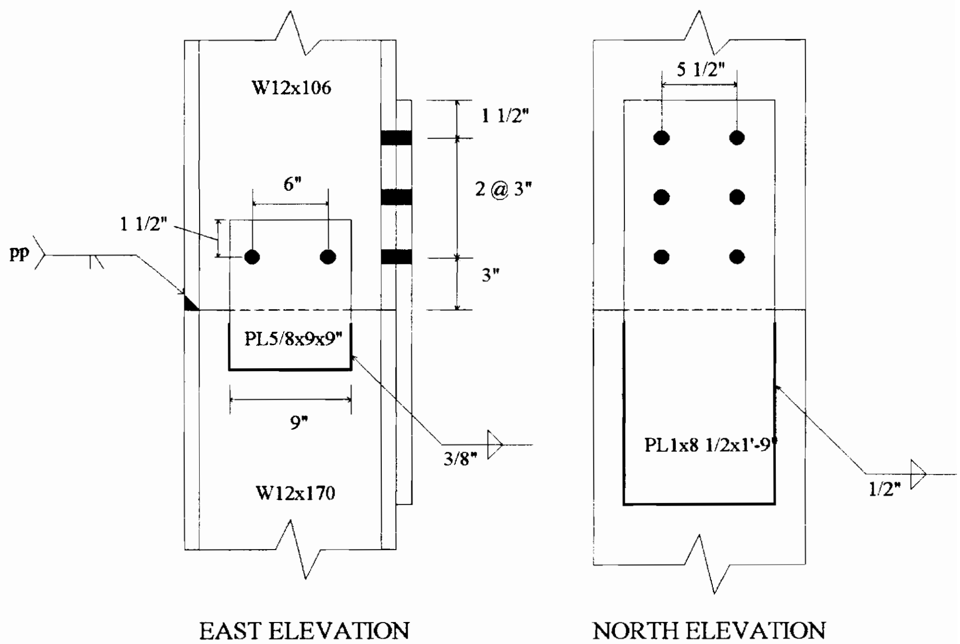
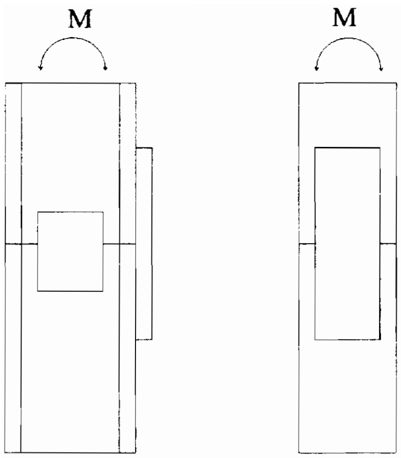


Figure 3.23

Show: All limit states that apply.



Assume C1 takes bearing stress from C2.

Splice plates are stressed by wind loads
(tensile bearing stresses) - Case I and II.

Limit States:

1. Column Bearing Stress

LRFD J8.1

$\phi R_n = \phi 2.0 F_y A_{pb}) = 0.75(2.0)(36)(31.2) = 1685^k$

For Flange Plate (Case I):

2. Bolt Shear

$\phi R_n = 0.75(48)(0.6013)(6) = 129.9^k$

3. Plate Tension Yield

$\phi T_n = 0.9(36)(8.5 \text{ x } 1.0) = 275.4^k$

4. Plate Tension Rupture

$\phi T_n = \phi F_u A_e$ where, $A_e = \min \left| \begin{array}{l} 0.85A_g \\ A_g - \Sigma A_{hole} \end{array} \right.$

$$A_e = \min \left| \begin{array}{l} 0.85(8.5 \text{ x } 1.0) = 7.2 \text{ in}^2 \\ (8.5 \text{ x } 1.0) - 2(7/8 + 1/8)1.0 = 6.5 \text{ in}^2 \end{array} \right.$$

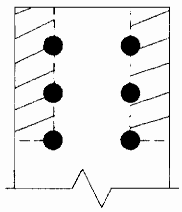
$\therefore \phi T_n = 0.75(58)(6.5) = 282.8^k$

5. Plate Bearing / Tear-out

edge $> 1.5d_b$ & spacing $= 3 > 3d_b$ $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 6[0.75(2.4)(58)(7/8)(1.0)] = 548.1^k$

6. Flange Plate Block Shear



Shear Rupture:

$0.6F_uA_{nv} = 0.6(58)[7.5 - 2.5(7/8 + 1/8)](1.0)2 = 348^k$

Tension Rupture:

$F_uA_{nt} = 58[1.5 - 0.5(7/8 + 1/8)](1.0)2 = 116^k$

Shear Rupture > Tension Rupture $\therefore$ Shear Rupture controls

Tension Yield: $F_yA_{gt} = 36(1.5 \times 1)2 = 108^k$

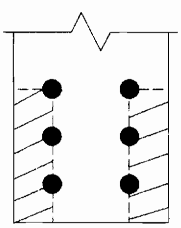
$\therefore \ \phi T_n = 0.75(348 + 108) = 342^k$

7. Column Flange Bearing / Tear-out

edge $> 1.5d_b$ & spacing $= 3 > 3d_b$ $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4F_ud_bt = 6[0.75(2.4)(58)(7/8)(1.56)] = 855^k$

8. Column Flange Block Shear



Shear Rupture:

$0.6F_uA_{nv} = 0.6(58)[9 - 2.5(7/8 + 1/8)](0.99)2$
$= 447.9^k$

Tension Rupture:

$F_uA_{nt} = 58[3.35 - 0.5(7/8 + 1/8)](0.99)2 = 327.3^k$

Shear Rupture > Tension Rupture $\therefore$ Shear Rupture controls

Tension Yield: $F_yA_{gt} = 36(3.35 \times 0.99)2 = 238.8^k$

$\therefore \ \phi T_n = 0.75(447.9 + 238.8) = 515^k$

9. Weld Rupture (1/2" fillet)

$\phi R_n = 8(1.392)[8.5 + 2(10.5 - 2(1/2))] = 306.2^k$

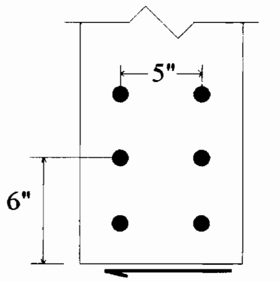
10. Plate Tension Rupture (welded portion)

$\phi R_n = 0.75(58)[0.85(8.5 \times 1.0)] = 314.3^k$

For Flange Plate (Case II):

11. Bolt Shear

Must consider eccentricity, using LRFD Table XIV



Assume V_u at splice

$b = 5.5$ in
$x_o = 6$ in $\Rightarrow$ Interpolating, $C = 2.81$
$n = 2$

$\phi R_n = 2.81(0.75)(48)(0.6013) = 60.8^k$

12. Plate Shear Yield

$\phi R_n = 0.9(0.6 \times 36)(8.5 \times 1.0) = 165.2^k$

13. Plate Shear Rupture

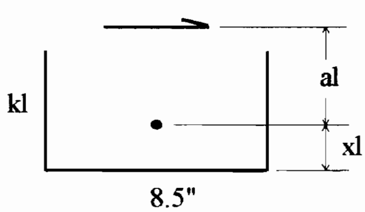
$\phi R_n = 0.75(0.6 \times 58)(8.5 - 2(7/8 + 1/8))(1.0) = 169.7^k$

14. Plate Bearing / Tear-out
edge $> 1.5d_b$ & spacing $= 3 > 3d_b$
$\therefore$ Only bearing applies, and consider eccentricity.

$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 2.81[0.75(2.4)(58)(7/8)(1.56)] = 256.7^k$

15. Weld Rupture ($t_{weld} = 1/2$ in)

Using LRFD Table XXII and the following values:



$kl = 10.5 - 2w = 10.5 - 2(0.5) = 9.5$ in.

$xl = \dfrac{2[9.5(4.75)]}{8.5 + 2(9.5)} = 3.28$ in.

$al = 10.5$ in. $- xl = 7.22$ in.

$a = \dfrac{al}{l} = \dfrac{7.22 \text{ in.}}{8.5 \text{ in.}} = 0.85$

$k = \dfrac{kl}{l} = \dfrac{9.5 \text{ in.}}{8.5 \text{ in.}} = 1.12$

Interpolation gives $C = 2.26$

$\therefore \phi R_n = P_u = C_1 C D l = (1)(2.26)(8)(8.5) = 153.7^k$

16. Column Flange Strength at Weld

Yield strength per inch of column = $\phi(0.6F_y t_w)$

$$= 0.9(0.6)(36)(1.56) = 30.3^{k/in.}$$

Strength per inch of weld = $1.392(8) = 11.14^{k/in.}$

$30.3 > 11.14$ $\therefore$ web is adequate for full weld

For Web Plate (Case I):

17. Bolt Shear
Must consider eccentricity, using LRFD Table X

$b = 6$ in
$x_o = 3$ in. $\Rightarrow$ Interpolating, $C = 1.39$
$n = 2$

$\phi R_n = 1.39(0.75)(48)(0.6013) = 30.1^k$

18. Plate Shear Yield

$\phi R_n = 0.9(0.6 \times 36)(9 \times 5/8) = 109.4^k$

19. Plate Shear Rupture

$\phi R_n = 0.75(0.6 \times 58)(9 - 2(7/8 + 1/8))(5/8) = 114.2^k$

20. Plate Bearing / Tear-out
edge $> 1.5d_b$ & spacing $= 3 > 3d_b$
$\therefore$ Only bearing applies, and consider eccentricity.

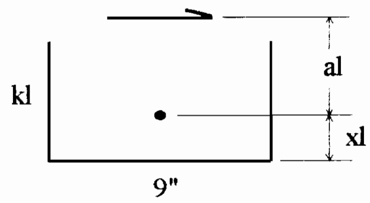
$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 1.39[0.75(2.4)(58)(7/8)(5/8)] = 79.4^k$

21. Column Web Bearing / Tear-out
edge $> 1.5d_b$ & spacing $= 3 > 3d_b$ $\therefore$ Only bearing applies.

$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 1.39[0.75(2.4)(58)(7/8)(0.61)] = 77.5^k$

22. Weld Rupture (t_{weld} = 3/8 in)

Using LRFD Table XXII and the following values:



$$kl = 4.5 - 2(3/8) = 3.75 \text{ in.}$$

$$xl = \frac{2[3.75(1.875)]}{9 + 2(3.75)} = 0.85 \text{ in.}$$

$$al = 4.5 \text{ in.} - xl = 3.65 \text{ in.}$$

$$a = \frac{al}{l} = \frac{3.652 \text{ in.}}{9 \text{ in.}} = 0.40$$

$$k = \frac{kl}{l} = \frac{3.75 \text{ in.}}{9 \text{ in.}} = 0.42$$

Interpolation gives C = 1.77

$\therefore\ \phi R_n = P_u = C_1 C D l = (1.0)(1.77)(6)(9) = 95.6^k$

23. Column Web Strength at Weld

Yield strength per inch of column = $\phi(0.6F_y t_w)$

$$= 0.9(0.6)(36)(0.96) = 18.66^{k/in.}$$

Strength per inch of weld = $1.392(6) = 8.35^{k/in.}$

$18.66 > 8.35$ $\therefore$ web is adequate for full weld

For Web Plate (Case II):

24. Bolt Shear

$\phi R_n = 0.75(48)(0.6013)(2) = 43.3^k$

25. Plate Tension Yield

$\phi T_n = 0.9(36)(9 \text{ x } 5/8) = 182.3^k$

26. Plate Tension Rupture

$\phi T_n = \phi F_u A_e$ where, $A_e = \min \left| \begin{array}{l} 0.85A_g \\ A_g - \Sigma A_{hole} \end{array} \right.$

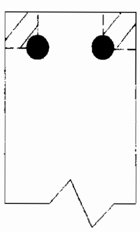
$A_e = \min \left| \begin{array}{l} 0.85(9 \times 5/8) = 4.78 \text{ in}^2 \\ (9 \times 5/8) - 2(7/8 + 1/8)(5/8) = 4.375 \text{ in}^2 \end{array} \right.$

$\therefore \phi T_n = 0.75(58)(4.375) = 190.3^k$

27. Plate Bearing / Tear-out
edge $> 1.5d_b$ & spacing $= 3 > 3d_b$ $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 2[0.75(2.4)(58)(7/8)(5/8)] = 114.2^k$

28. Flange Plate Block Shear



Shear Rupture:
$0.6F_u A_{nv} = 0.6(58)[1.5 - 0.5(7/8 + 1/8)](5/8)2 = 43.5^k$

Tension Rupture:
$F_u A_{nt} = 58[1.5 - 0.5(7/8 + 1/8)](5/8)2 = 50.75^k$

Tension Rupture > Shear Rupture $\therefore$ Tension Rupture controls

Shear Yield: $0.6F_y A_{gt} = 0.6(36)(1.5 \times 5/8)2 = 40.5^k$

$\therefore \phi T_n = 0.75(50.75 + 40.5) = 68.4^k$

29. Column Web Bearing / Tear-out
edge $> 1.5d_b$ & spacing $= 3 > 3d_b$ $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4F_u d_b t = 2[0.75(2.4)(58)(7/8)(0.61)] = 111.4^k$

30. Column Web Block Shear

Will not govern by inspection

31. Weld Rupture (3/8" fillet)

$\phi R_n = 6(1.392)[9 + 2(4.5 - 2(3/8 \times 2))] = 137.8^k$

32. Plate Tension Rupture (welded portion)

$\phi R_n = 0.75(58)[0.85(9 \times 5/8)] = 208^k$

For Partial Penetration Groove Weld:

33. Weld Strength

Using LRFD Table J2.3 $\Rightarrow$ min t_e = 3/8 in.

Assume 3/4 in. chamfer

from Table J2.1, $t_e = 3/4 - 1/8 = 5/8$ in.

$\phi R_n = 0.9(5/8)(36) = 20.25^{k/in} \Rightarrow \phi R_n = 20.25(12.2) = 247.1^k$ (compression)

$\phi R_n = 0.8(5/8)(0.6 \times 70) = 21^{k/in} \Rightarrow \phi R_n = 21(12.2) = 256.2^k$ (shear)

$\therefore$ Bolt Shear in Web Plate controls connection design $\Rightarrow \phi R_n = 30.1^k$

3.4.2 Bearing Connections

Example 1. **Seated Beam Connection** - Refer to beam B6 into column C2 connection. (Fig. 3.2, 3.5)

Given: A W16x40 beam framing into a W12x106 column connection.
Unstiffened angle, seated beam connection.
L4x4x3/8x6 1/2" (seat angle)
L4x4x3/8x6" (erection angles)
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.24 below.

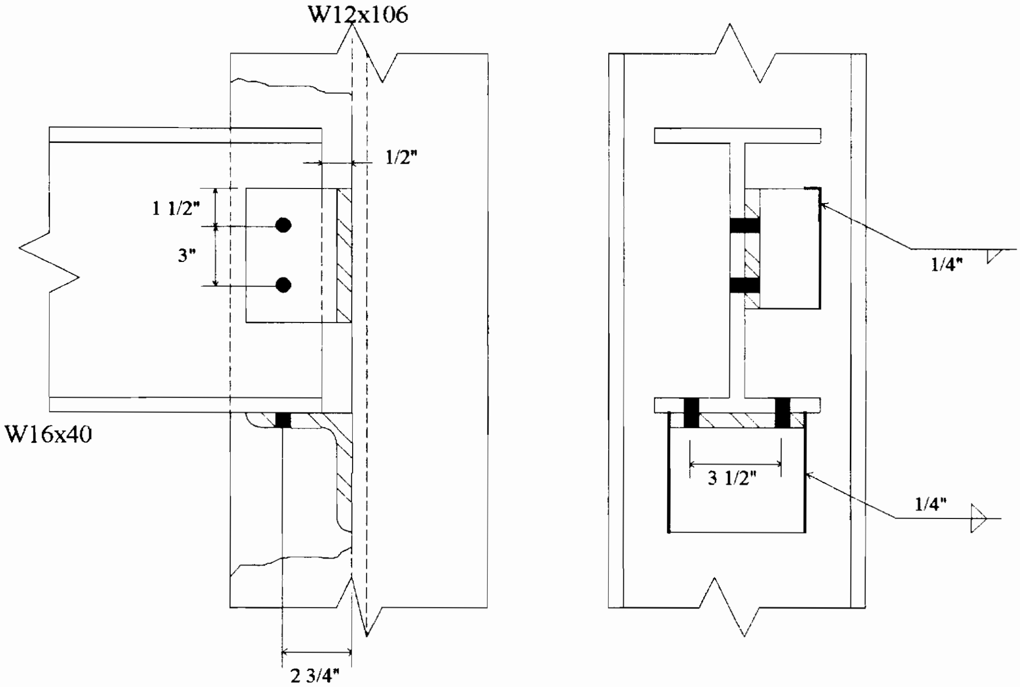


Figure 3.24

Show: All limit states that apply.

Limit States:

Note: Refer to "Design Loads for Seated-beam in LRFD." (Garrett, *et al* 1986)

1. Beam Gross Shear

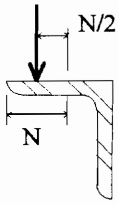
$$\phi R_n = 0.9(0.6x36)(16.01 \text{ x } 0.305) = 94.9^k$$

2. Beam Net Shear

$$\phi R_n = 0.75(0.6x58)[16.01 - 2(7/8 + 1/8)](0.305) = 111.5^k$$

3. Beam Web Local Yielding

Use N as shown below.



Conservatively, use N = angle leg - 3/4 = 3.25 in.

$$\phi R_n = \phi(N + 2.5k)F_y t_w$$

$$= 1.0[3.25 + (2.5 \text{ x } 1.1875)](36 \text{ x } 0.305) = 68.3^k$$

4. Beam Local Web Crippling

$$N / 2 = 1.625 < d/2$$
$$N / d = 1.625 / 16.01 = 0.203 > 0.2$$

$$\phi R_n = \phi 68 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\phi R_n = (0.75)68(0.305)^2\left[1 + \left(\frac{4(3.25)}{16.01} - 0.2\right)\left(\frac{0.305}{0.505}\right)^{1.5}\right]\sqrt{\frac{36(0.505)}{0.305}}$$

$$\phi R_n = 47.1^k$$

5. Angle Shear Yield

$$\phi R_n = 0.9(0.6 \text{ x } 36)(6.5 \text{ x } 3/8) = 47.4^k$$

6. Flexural Strength of Angle

$$S = \frac{l^2 t}{6} = \frac{(6.5)^2(3/8)}{6} = 2.64 \text{ in}^2$$

$$e = \frac{N}{2} + \frac{3}{4} - \left(t + \frac{3}{8}\right)$$

$$e = \frac{3.25}{2} + \frac{3}{4} - \left(\frac{3}{8} + \frac{3}{8}\right) = 1.625 \text{ in.}$$

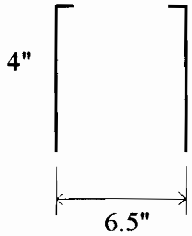
$$\phi V_n = 0.9 F_y S_g / e,$$

$$\therefore \phi V_n = \frac{0.9(36)(2.64)}{(1.625)} = 52.7^k$$

7. Column Flange Yield at Weld

$\phi R_n = 0.9(0.6 \times 36)(4 \times 3/8)2 = 58.3^k$

8. Weld Rupture (1/4" fillet)



Neglect weld returns.

$e_w = N/2 + 3/4 = 2.375$ in.

Using LRFD Table XVIII and
$kl = 6.5$ in. $\Rightarrow k = 1.625$
$al = 2.375$ in. $\Rightarrow a = 0.69$ $\therefore C = 1.87$

$\phi R_n = P_u = 1.0(1.87)(4)(4) = 30^k$

9. Column Flange Yield at Weld

$\phi R_n = 0.9(0.6 \times 36)(4 \times 0.61)2 = 94.9^k$

For erection angle:

10. Angle Shear Yield

$\phi R_n = 0.9(0.6 \times 36)(6 \times 3/8) = 43.7^k$

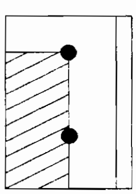
11. Angle Shear Rupture

$\phi R_n = 0.75(0.6 \times 58)[6 - 2(7/8 + 1/8)](3/8) = 39.2^k$

12. Angle Bearing / Tear-out
edge $> 1.5d_b$ & spacing $= 3 > 3d_b$ $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 2[0.75(2.4)(58)(7/8)(0.375)] = 68.5^k$

13. Angle Block Shear



Shear Rupture:
$0.6 F_u A_{ns} = 0.6(58)[6 - 1.5(7/8 + 1/8)](0.375) = 58.7^k$

Tension Rupture:
$F_u A_{nt} = 58(1.25 - 0.5(7/8 + 1/8))(0.375) = 16.3^k$

Shear Rupture > Tension Rupture $\therefore$ Shear Rupture controls

Tension Yield: $F_y A_{gt} = 36(1.25 \times 3/8) = 16.9^k$

$\therefore \phi V_n = [0.75(58.7 + 16.9)] = 56.7^k$

14. Bolt Shear

$$\phi R_n = 0.75(48))(0.6013)2 = 43.3^k$$

15. Flexural Yield of Angle

$$\phi V_n = 0.9F_y S_g / e, \qquad S_g = \frac{l^2 t}{6} = \frac{(6)^2(3/8)}{6} = 2.25 \text{ in}^2$$

$$\therefore \phi V_n = \frac{0.9(36)(2.25)}{(2.75)} = 26.5^k$$

16. Flexural Rupture of Angle

$$\phi V_n = 0.75F_y S_{net} / e$$

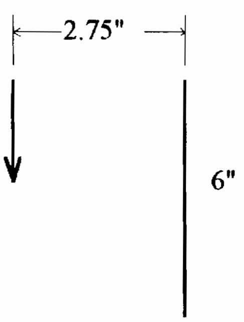
$$S_{net} = S_g - \frac{s^2 N_b (N_b^2 - 1)[t(d_b + 1/8)]}{6l}$$

$$= 2.25 - \frac{(3)^2(2)[0.375(7/8 + 1/8)]}{6(6)} = 1.69 \text{ in}^2$$

$$\therefore \phi V_n = \frac{0.75(58)(1.69)}{(2.75)} = 26.7^k$$

17. Weld Shear Strength ($t_{weld} = 1/4$ in)

Using LRFD Table XVIII and the following values:



$$al = = 2.75 \text{ in.}$$

$$a = \frac{al}{l} = \frac{2.75 \text{ in.}}{6 \text{ in.}} = 0.46$$

$$kl = 0, \; k = 0$$

Interpolation gives C = 1.399

$$\therefore \phi R_n = P_u = C_1 C D l = (1)(1.399)(4)(6) = 33.6^k$$

$\therefore$ Weld Rupture controls the connection design $\Rightarrow \phi R_n = 30^k$

(Ignoring the erection angle limit states)

Example 2. **Beam Bearing Connection** - Refer to column C3 bearing on beam B4. (Fig. 3.3, 3.4)

Given: A 4" ϕ STD Pipe column bearing onto a W18x60 beam connection.
Bearing stiffeners (half-depth)
2-PL3/8x3 1/2x8 1/2"
All steel is A36.
See Fig. 3.25 below.

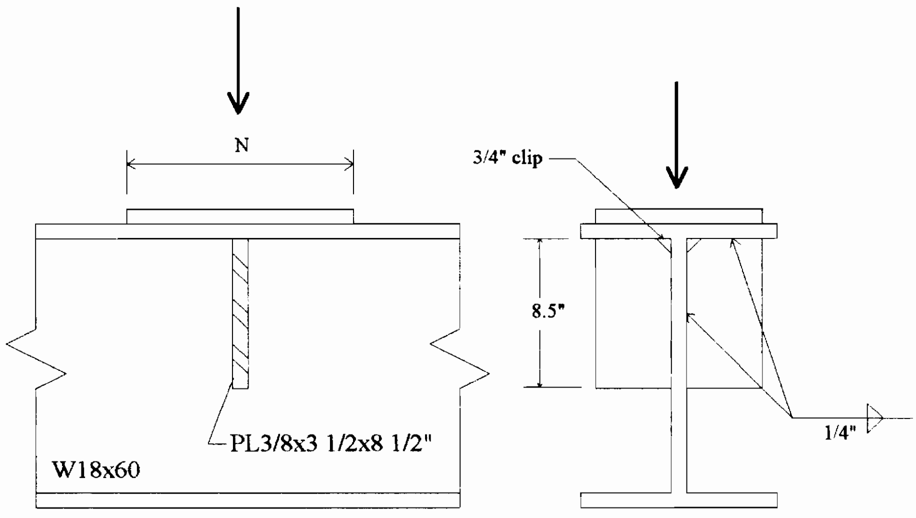


Figure 3.25

Show: All limit states that apply.

Limit States:

1. Local Web Yielding
Load distance > d from end, N = 10.5 in.

$$\therefore\ \phi R_n = 1.0[5(1.375) + 10.5](36)(0.415) = 260.6^k$$

2. Local Web Crippling
Load distance > d/2,

$$\phi R_n = \phi 135 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\phi R_n = (0.75)135(0.415)^2\left[1 + 3\left(\frac{(10.5)}{18.24}\right)\left(\frac{0.415}{0.695}\right)^{1.5}\right]\sqrt{\frac{36(0.695)}{0.415}}$$

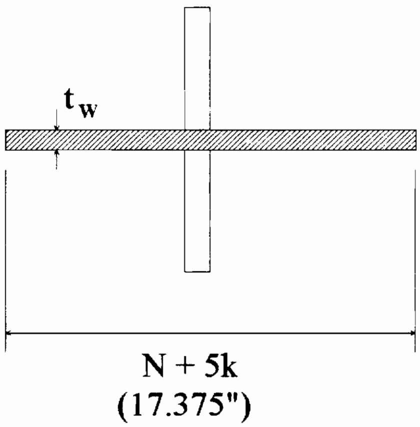
$$\phi R_n = 243.3^k$$

3. Sidesway Web Buckling (LRFD K1.5)

$$\frac{h/t_w}{l/b_f} = \left(\frac{38.7}{4(12)/7.555}\right) = 6.1 \geq 7 \quad \therefore \text{ will not occur.}$$

4. Stiffener Capacity 2-PL3/8x3 1/2x8 1/2" (half depth)

$$\frac{b}{t} = \frac{3.5}{0.375} = 9.33 \leq \frac{95}{\sqrt{F_y}} = 15.833 \quad \therefore \text{ ok}$$



$I = 1/12(3/8)(7.415)^3 = 12.74 \text{ in}^4$

$A = 2(3.5 \times 3/8) + 17.375(0.415) = 9.84 \text{ in}^2$

$$r = \sqrt{\frac{12.74}{9.84}} = 1.14 \text{ in.}$$

$kL = 0.75(8.5) = 6.375 \text{ in.}$

$kL/r = 6.375/1.14 = 5.6 \Rightarrow \phi F_{cr} = 30.55 \text{ ksi}$

$\therefore \phi P_n = 30.55(9.84) = 300.6^k$

5. Weld Strength

$\phi R_n = 4 \times 4(1.392)(8.5 - 0.75) = 172.6^k$

$\therefore$ Weld Strength controls the connection design $\Rightarrow \phi R_n = 172.6^k$

Example 3. **Beam Bearing Connection** - Refer to beam B8 bearing on column C3. (Fig. 3.3, 3.4)

Given: A W16x31 beam bearing onto a 4" ϕ STD Pipe column connection. All steel is A36. See Fig. 3.26.

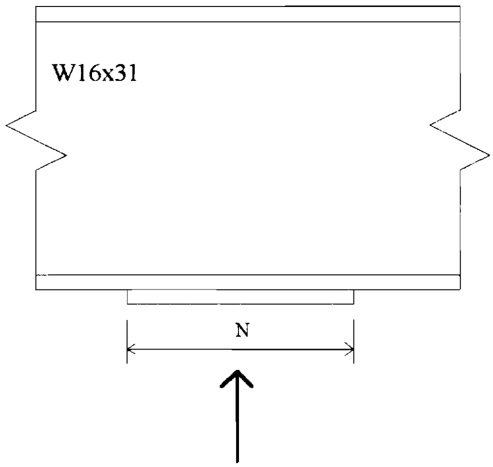


Figure 3.26

Show: All limit states that apply.

Limit States:

1. Local Web Yielding
   Load distance < d from end, N = 10.5 in.

   $\therefore\ \phi R_n = 1.0[2.5(1.125) + 10.5](36)(0.275) = 131.8^k$

2. Local Web Crippling
   Load distance > d/2,

$$\phi R_n = \phi 135 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\phi R_n = (0.75)135(0.275)^2\left[1 + 3\left(\frac{10.5}{15.88}\right)\left(\frac{0.275}{0.44}\right)^{1.5}\right]\sqrt{\frac{36(0.44)}{0.275}}$$

$$\phi R_n = 115.1^k$$

$\therefore$ Web Crippling controls the connection design $\Rightarrow \phi R_n = 115.1^k$

Example 4. **Column Base-Plate Connection** - Refer to column C3 connection. (Fig. 3.3, 3.4)

Given: A W16x31 beam bearing onto a 4" ϕ STD Pipe column connection.
PL5/8x5 1/2x10 1/2" (top)
PL5/8x7 1/2x10 1/2" (bottom)
All steel is A36.
See Fig. 3.27.

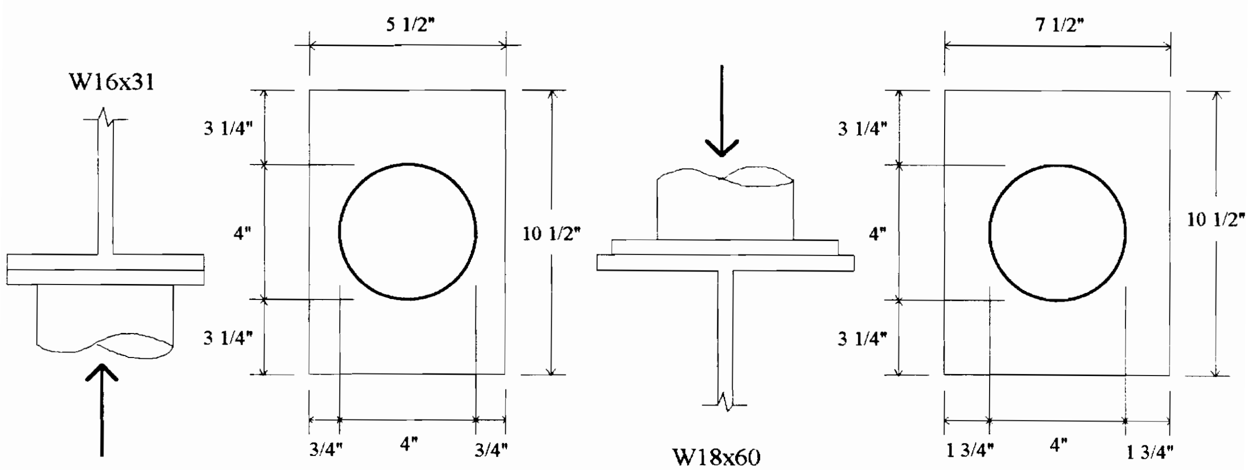


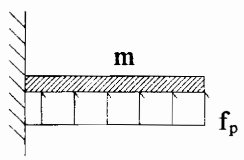
Figure 3.27

Show: All limit states that apply.

Limit States:

For Top Plate:

1. Plate Bearing



$$m = \frac{b}{2} - k = \frac{5.5}{2} - 1.125 = 1.625 \text{ in.}$$

$$Z_x = \frac{(1")t_p^2}{4} = \frac{(1")(0.625)^2}{4} = 0.0977 \text{ in}^3$$

$$M_u = \phi F_y Z_x = 0.9(36)(0.0977) = 3.16^{k\text{-in}}$$

$$f_p = \frac{2M_u}{m^2} = \frac{2(3.16)}{(1.625)^2} = 2.4 \text{ ksi}$$

$$f_p = \frac{P_u}{NB}$$

$$\therefore \phi R_n = f_p NB = 2.4(5.5)(10.5) = 138.6^k$$

2. Plate Bending

$m = 3.25$ in (critical)

$\therefore M_u = 3.16^{k\text{-}in} \Rightarrow f_p = 2(3.16)/(3.25)^2 = 0.60$ ksi

$\phi R_n = 0.60(5.5)(10.5) = 34.7^k$

For Bottom Plate:

3. Plate Bearing

$$m = \frac{b}{2} - k = \frac{7.5}{2} - 1.375 = 2.375 \text{ in.}$$

$$Z_x = \frac{(1")t_p^2}{4} = \frac{(1")(0.625)^2}{4} = 0.0977 \text{ in}^3$$

$$M_u = \phi F_y Z_x = 0.9(36)(0.0977) = 3.16^{k\text{-}in}$$

$$f_p = \frac{2M_u}{m^2} = \frac{2(3.16)}{(2.375)^2} = 1.12 \text{ ksi}$$

$$f_p = \frac{P_u}{NB}$$

$$\therefore \phi R_n = f_p NB = 1.12(7.5)(10.5) = 88.2^k$$

4. Plate Bending

$m = 3.25$ in. (critical) and $f_p = 0.60$ ksi

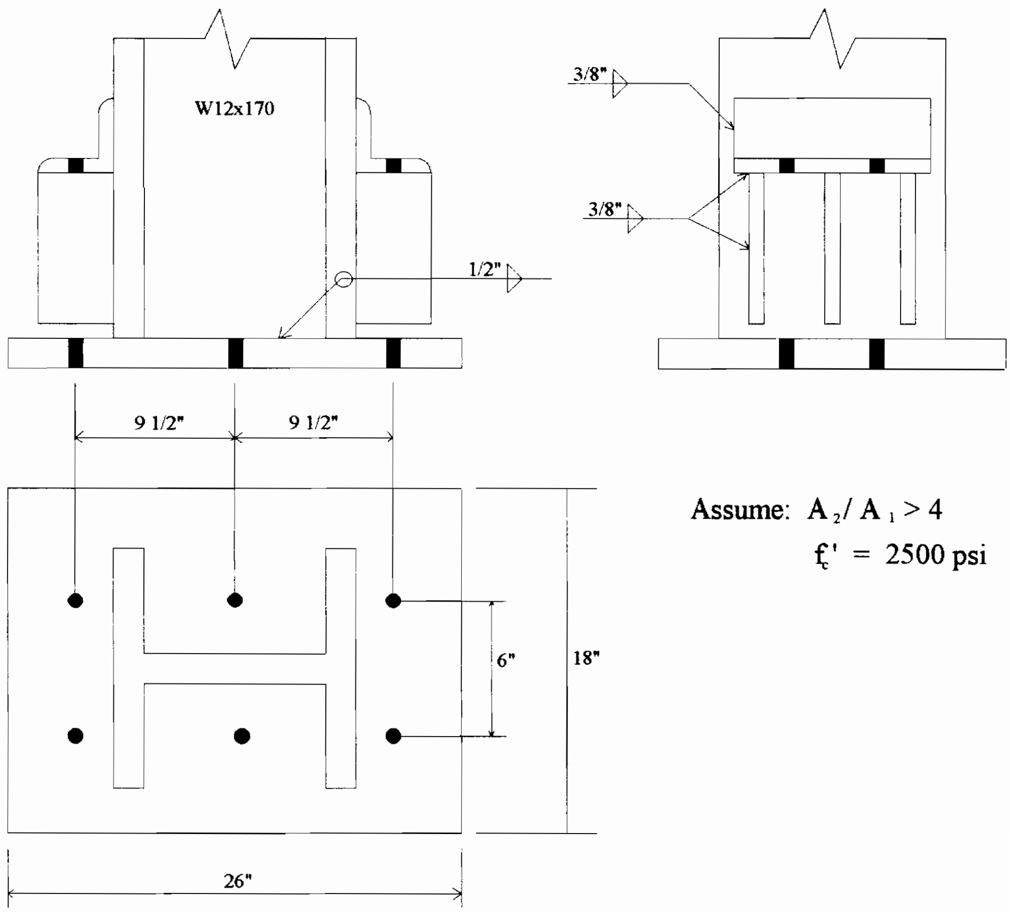
$\phi R_n = 0.60(7.5)(10.5) = 47.3^k$

$\therefore$ Plate Bending (Top Plate) controls the connection $\Rightarrow \phi R_n = 34.7^k$

Example 5. **Moment Resistant Column Base-Plate Connection**

- Refer to column C1 base-plate connection. (Fig. 3.1, 3.4, 3.5)

Given: A W12x170 column bearing connection.
PL 1 1/2x18x2'-2"
2-L4x4x1'-0", 6-PL 3/4x4x8 and 6 1 1/4" ϕ Anchor bolts.
All steel is A36.
See Fig. 3.28 below.

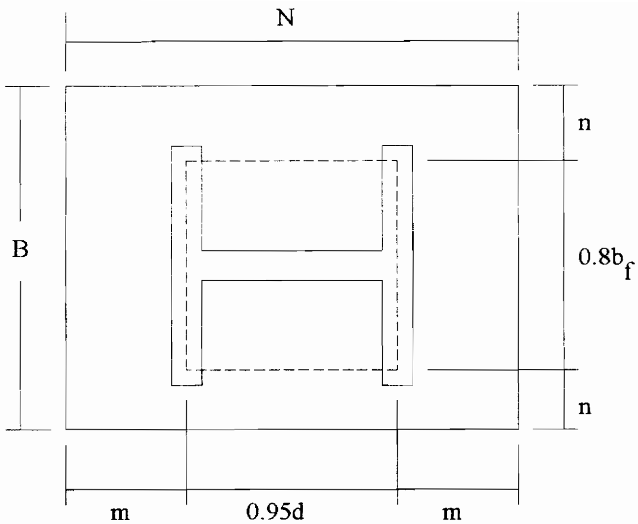


Assume: $A_2 / A_1 > 4$

$f_c' = 2500$ psi

Figure 3.28

Show: All limit states that apply.



Limit States:

Axial Load Only:

1. Concrete Crushing

Since $A_2 / A_1 > 4$,

$$\phi P_p = \phi 0.85 f'_c A_1 \sqrt{4} = 0.6(0.85)(2.5)[18 \times 26]\sqrt{4} = 1193.4^k$$

2. Plate Bending

n = 3.972 in. < m = 6.3358 in. ∴ m controls

$t_p = 1.5$ in.

$$Z_x = \frac{(1")t_p^2}{4} = \frac{(1")(1.5)^2}{4} = 0.5625 \text{ in}^3$$

$$M_u = \phi F_y Z_x = 0.9(36)(0.5625) = 18.225^{k-in}$$

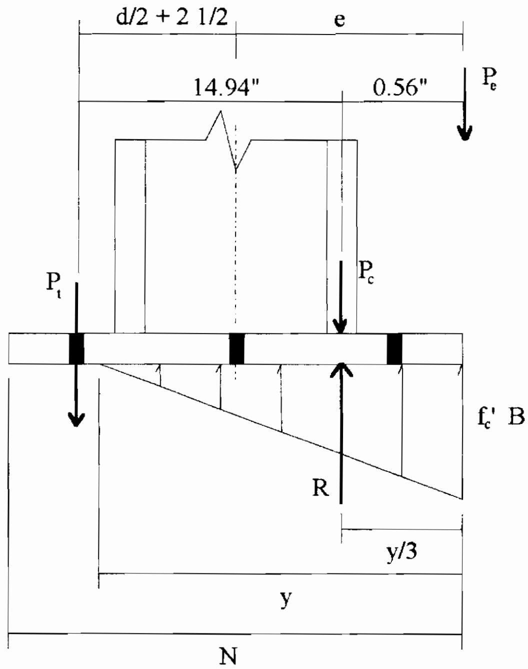
$$f_p = \frac{2M_u}{m^2} = \frac{2(18.225)}{(6.3358)^2} = 0.91 \text{ ksi}$$

$$f_p = \frac{P_u}{NB}$$

$$\therefore \phi R_n = f_p NB = 0.91(18)(26) = 425.8^k$$

Consider Eccentric Load (Moment):

Note: Refer to *Column Base Plates* (DeWolf *et al* 1991).



Assume $P_u = 100^k$, $M_u = 50^{k\text{-}ft}$

$\therefore$ e = 50(12) / 100 = 6 in.

Load on Bolts:

P_t = 100(0.56) / 14.94 = 3.75^k

Assume 1 1/4" ϕ anchor bolts, A36 $\Rightarrow$ $A_b = 1.23$ in^2 (use 2 anchor bolts)

$\therefore$ $f_b = F_b / 2A_b = 3.75 / 2(1.23) = 1.52$ ksi << 36 ksi, ok

Compressive Flange Resultant = R = $P_c + P_t$ = 103.75^k

$y = 2(P_c + P_t) / f_cB$, $f_c = 0.25f_c'$, Let B = 18 in. and $f_c' = 2500$ psi

$\therefore$ y = 2(103.75) / 0.25(2.5)(18) = 18.4 in.

y/3 = 6.1 in.

$\therefore$ N = 2(6.1) + (14.03 - 1.56) = 24.7 in. $\Rightarrow$ Use N = 26 in.

Therefore,

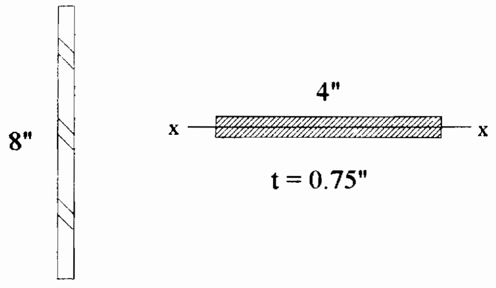
Base-plate dimensions should be: 26 in. x 18 in.
(base-plate is adequate to resist moment)

For Angles and Stiffener Plates:

1. Tension Yield of Anchor Bolts

$\phi R_n = 0.9(36)(1.23)2 = 79.7^k$

2. Compressive Capacity of Stiffener Plates



$\frac{b}{t} \le \frac{95}{\sqrt{F_y}}$, ok

$I_x = 1/12(4)(0.75)^3 = 0.1406 \text{ in}^4$

$A_g = 4(0.75) = 3 \text{ in}^2$

$r_x = \sqrt{\frac{I_x}{A_g}} = \sqrt{\frac{0.1406}{3}} = 0.05 \text{ in.}$

use $k = 0.75$ $\quad kL/r = 0.75(8)/0.05 = 120 \Rightarrow \quad \phi F_{cr} = 14.34$ ksi

$\therefore \ \phi R_n = 14.34(3) = 129.1^k$

3. Weld Rupture (at Column Flange) (3/8" fillets)

$\phi R_n = 2(6)(1.392)(8 - 0.75)\text{x}3 = 363.3^k$

4. Plate Yield at Weld

$\phi R_n = 0.9(0.6 \text{ x } 36)[8 - 0.75](0.75)2\text{x}3 = 634.2^k$

5. Column Flange Yield at Weld

$\phi R_n = 0.9(0.6 \text{ x } 36)[8 - 0.75](1.56)2\text{x}3 = 1319^k$

6. Shear Yield of Angle

$\phi R_n = 0.9(0.6 \text{ x } 36)(12 \text{ x } 1/2) = 116.6^k$

7. Weld Rupture (at Column Flange) (3/8" fillets)

$\phi R_n = (6)(1.392)[12(2) + 2(4)] = 267.3^k$

8. Angle Yield at Weld

$\phi R_n = 0.9(0.6 \text{ x } 36)[12(2) + 2(4)](0.5) = 311^k$

9. Weld Strength (at Column Flange) (3/8" fillets)

$\phi R_n = 6(1.392)(4 - 0.75)2\text{x}3 = 162.9^k$

$\therefore$ Angle Shear Yield controls connection design $\Rightarrow \phi R_n = 116.6^k$

3.4.3 Semi-Rigid Flexible Wind Connection

<u>**Example 1.**</u> **Seated Beam (Flexible Wind) Connection** - Refer to beam B5 into column C2 connection. (Fig. 3.3, 3.5)

<u>Given:</u> A W16x36 beam framing into a W12x106 column connection.
Unstiffened angle, seated beam/flexible wind connection.
L6x6x3/4x5 1/2" (seat angle)
L4x4x3/8x6" (top flange angle)
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.29 below.

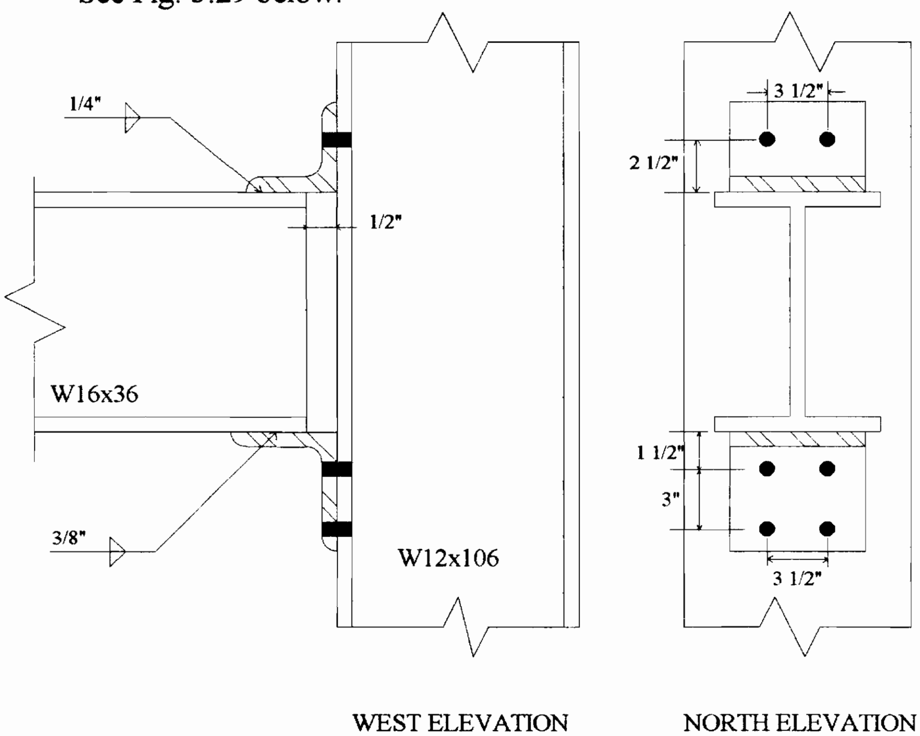


Figure 3.29

<u>Show:</u> All limit states that apply.

Note: Refer to LRFD Vol. II. Flexible Wind Connections (AISC 1992).

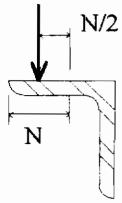
<u>Limit States:</u>

1. <u>Beam Gross Shear</u>

$\phi R_n = 0.9(0.6x36)(15.86x0.295) = 91.0^k$

2. Beam Web Local Yielding

Use N as shown below.



Conservatively, use N = angle leg - 3/4 = 5.25 in.

$$\phi R_n = \phi(N + 2.5k)F_y t_w$$

$$= 1.0[5.25 + (2.5 \times 1.125)](36x0..295) = 85.6^k$$

3. Beam Local Web Crippling

$N / 2 = 2.625 < d/2$

$N / d = 5.25 / 15.86 = 0.33 > 0.2$

$$\phi R_n = \phi 68 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\phi R_n = (0.75)68(0.295)^2 \left[1 + \left(\frac{4(5.25)}{15.86} - 0.2\right)\left(\frac{0.295}{0.43}\right)^{1.5}\right]\sqrt{\frac{36(0.43)}{0.295}}$$

$$\phi R_n = 42.2^k$$

For Bottom Angle:

4. Angle Shear Yield

$$\phi R_n = 0.9(0.6 \times 36)(5.5 \times 3/4) = 80.2^k$$

5. Flexural Strength of Angle

$$S = \frac{l^2 t}{6} = \frac{(5.5)^2(3/4)}{6} = 3.78 \text{ in}^2$$

$$e = \frac{N}{2} + \frac{3}{4} - \left(t + \frac{3}{8}\right)$$

$$e = \frac{5.25}{2} + \frac{3}{4} - \left(\frac{3}{4} + \frac{3}{8}\right) = 2.25 \text{ in.}$$

$$\phi V_n = 0.9F_y S_g / e,$$

$$\therefore\ \phi V_n = \frac{0.9(36)(3.78)}{(2.25)} = 54.5^k$$

6. Angle Bearing / Tear-out
No edge distance concern & spacing = 3 > 3d_b $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 4[0.75(2.4)(58)(7/8)(0.75)] = 274.1^k$

7. Bolt Rupture (Shear and Tension)

Using LRFD Table J3.5 (for A325-N bolts)

$F_t = 117 - 1.9 f_v \leq 90$ ksi

$f_v \leq \phi F_v \Rightarrow$ set $f_v = \phi F_v = 0.75(48) = 36$ ksi

$\therefore F_t = 117 - 1.9(36) = 48.6 \leq 90$ ok

$\phi R_n = \sum \phi F_t A_b = 0.75(48.6)(0.6013)4 = 87.7^k$

8. Column Flange Bearing / Tear-out
No edge distance concern & spacing = 3 > 3d_b $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 4[0.75(2.4)(58)(7/8)(0.99)] = 361.7^k$

9. Weld Rupture (Bottom Angle) (t_{weld} = 3/8 in)

$\therefore \phi R_n = (2)(1.392)(6)(6 - 3/4) = 87.7^k$

10. Angle Yield at Weld

$\phi R_n = 2 \times 0.9(0.6 \times 36)[6 - 3/4](3/4) = 153.1^k$

11. Beam Flange Yield at Weld

$\phi R_n = 2 \times 0.9(0.6 \times 36)[6 - 3/4](0.43) = 87.8^k$

For Top Angle:

12. Angle Shear Yield

$\phi R_n = 0.9(0.6 \times 36)(5.5 \times 3/8) = 40.1^k$

13. Angle Bearing / Tear-out
No edge distance concern & spacing = 3 > 3d_b $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 2[0.75(2.4)(58)(7/8)(0.375)] = 68.5^k$

14. Bolt Rupture (Shear and Tension)

Using LRFD Table J3.5 (for A325-N bolts)

$F_t = 117 - 1.9 f_v \leq 90$ ksi

$f_v \leq \phi F_v \Rightarrow$ set $f_v = \phi F_v = 0.75(48) = 36$ ksi

$\therefore\ F_t = 117 - 1.9(36) = 48.6 \leq 90 \quad$ ok

$\phi R_n = \sum \phi F_t A_b = 0.75(48.6)(0.6013)2 = 43.9^k$

15. Column Flange Bearing / Tear-out
No edge distance concern & spacing = 3 > $3d_b$ $\therefore$ Only bearing applies

$\phi R_n = \phi \sum_{bolts} 2.4 F_u d_b t = 2[0.75(2.4)(58)(7/8)(0.99)] = 180.9^k$

16. Weld Rupture (Top Angle) (t_{weld} = 1/4 in)

$\therefore\ \phi R_n = (2)(1.392)(4)[4 - 2(1/4)] = 39.0^k$

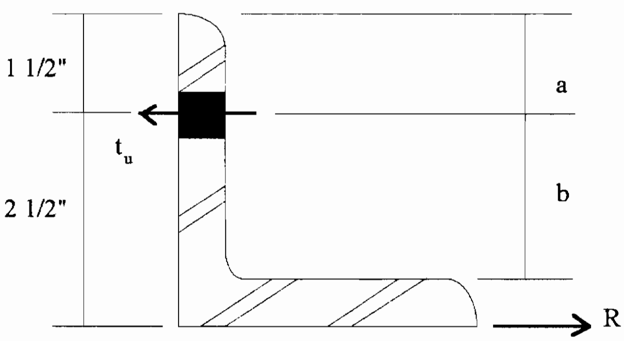
17. Angle Yield at Weld

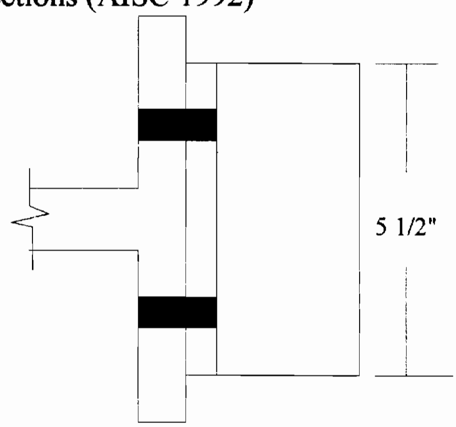
$\phi R_n = 2 \times 0.9(0.6 \times 36)[4 - 2(1/4)](3/8) = 51^k$

18. Beam Flange Yield at Weld

$\phi R_n = 2 \times 0.9(0.6 \times 36)[4 - 2(1/4)](0.43) = 58.5^k$

19. Prying Action See LRFD Vol. II - Hanger Connections (AISC 1992)





$\phi r_t = 0.75(90)(0.6013) = 40.6^k$

$p = 2.75$ in.
$b = 2.5 - t_a = 2.5 - 0.375 = 2.125$ in.
$b' = b - d_b/2 = 2.125 - 0.875/2 = 1.6875$ in.
$a = 1.5$ in.
$a' = a + d_b/2 = 1.5 + 0.875/2 = 1.9375$ in.
$\rho = b' / a' = 0.871$

$$\delta = 1 - \frac{d'}{p}, \text{ where } d' = \text{nominal } d_h$$

$$\delta = 1 - \frac{(7/8 + 1/16)}{2.75} = 0.6591$$

Assume $F_{fu} = 36.2^k$ (top angle controlling limit state)

$\therefore\ r_u = 36.2 / 2 = 18.1^k$

$$\phi Z_x F_y = 0.9\left(2.75\frac{(0.375)^2}{4}\right)36 = 3.13^{k\text{-}in}$$

$$M_{u1} = \frac{r_u b'}{1+\delta\alpha} = \phi Z_x F_y$$

$$\therefore\ \alpha = \frac{r_u b' - M_{u1}}{M_{u1}\delta} = \frac{18.1(1.6875) - 3.13}{3.13(0.6591)} = 13.3 > 1.0$$

$\therefore$ use $\alpha = 1.0$

$$t_u = r_u\left(\frac{\delta\alpha}{1+\delta\alpha}\rho + 1\right) = 18.1\left(\frac{0.6951(1.0)}{1+0.6951(1.0)}(0.871) + 1\right) = 24.4k$$

$\therefore$ Since $t_u < \phi r_t$, The connection is adequate for prying action.

$\therefore$ Top Angle Shear Yield Rupture controls the connection design
(for shear) $\Rightarrow \phi R_n = 40.1^k$

Weld Rupture (Top Angle) controls the connection design
(for moment) $\Rightarrow \phi R_n = 36.2^k$

3.4.4 Skewed Beam Framing Connection

Example 1. **Skewed Beam Framing Connection** - Refer to beam B9 framing into beams B5 and B8. (Fig. 3.2, 3.3, 3.6)

Given: A C9x15 channel framing into a W16x36 (B5) and a W16x31 (B8).
Skewed beam connection.
2-PL1/4x6x8" bent at 45°
Use 7/8 in. ϕ A325-N bolts.
All steel is A36.
See Fig. 3.30 below.

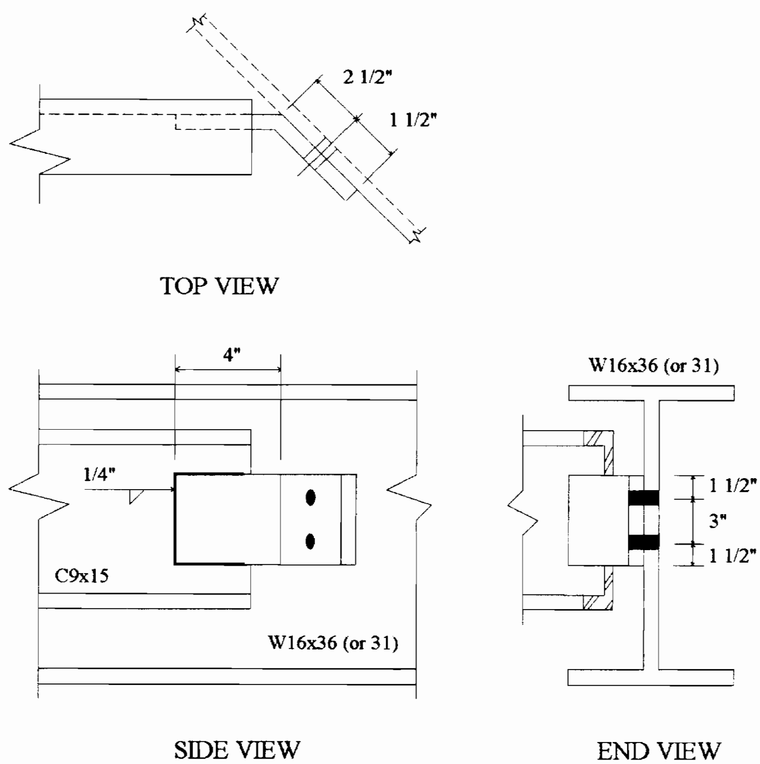


Figure 3.30

Show: All limit states that apply.

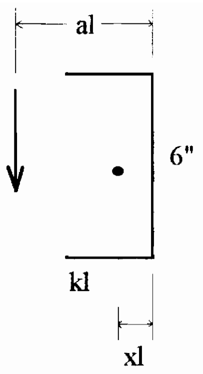
Limit States:

1. Channel Gross Shear

$$\phi V_n = \phi(0.6F_yA_w) = 0.9(0.6)(36)(9 \times 0.285) = 49.9^k$$

2. Weld Shear Strength (t_{weld} = 1/4 in)

Using LRFD Table XXII and the following values:



$$kl = 4 - \frac{1}{2} - 2(0.25) = 3 \text{ in.}$$

$$xl = \frac{2[3(1.5)]}{6 + 2(3)} = 0.75 \text{ in.}$$

$$al = 4 \text{ in.} - xl = 3.25 \text{ in.}$$

$$a = \frac{al}{l} = \frac{3.25 \text{ in.}}{6 \text{ in.}} = 0.54$$

$$k = \frac{kl}{l} = \frac{3 \text{ in.}}{6 \text{ in.}} = 0.5$$

Interpolation gives C = 1.64

$\therefore\ \phi R_n = P_u = C_1CDl = (1)(1.64)(4)(6) = 39.4^k$

3. Channel Web Strength at Weld

Yield strength per inch of channel= $\phi(0.6F_yt_w)$

$= 0.9(0.6)(36)(0.285) = 5.54^{k/in.}$

Strength per inch of weld $= 1.392(4) = 5.57^{k/in.}$

$5.54 < 5.57\ \therefore$ web is not adequate for full weld

$$\frac{5.54}{5.57}(39.4) = 39.2 \text{ k}$$

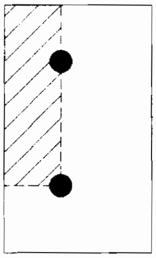
4. Plate Gross Shear

$\phi V_n = \phi(0.6F_yA_g) = 0.9(0.6)(36)(6 \times 0.25) = 29.2^k$

5. Plate Net Shear

$\phi V_n = \phi(0.6F_uA_{ns}) = 0.75(0.6)(58)[6 - 2(7/8 + 1/8)]0.25 = 26.1^k$

6. Plate Block Shear



Shear Rupture:

$0.6F_uA_{ns} = 0.6(58)[4.5 - 1.5(7/8 + 1/8)].25$
$= 26.1^k$

Tension Rupture:

$F_uA_{nt} = 58[(1.5 - 0.5(7/8 + 1/8)]0.25$
$= 14.5^k$

Shear Rupture > Tension Rupture
∴ Shear Rupture controls

Tension Yield: $F_yA_{gt} = 36(1.5 \times 0.25) = 13.5^k$

∴ $\phi V_n = 0.75(26.1 + 13.5) = 29.7^k$

7. Plate Bearing / Tear-out
edge = 1.25 in. > $1.5d_b$ & spacing = 3 > $3d_b$ ∴ Only bearing applies

Note: Use effective number of bolts, C = 1.03 from bolt shear limit state.

$\phi R_n = \phi\sum_{bolts} 2.4F_ud_bt = 1.03[0.75(2.4)(58)(7/8)(.25)] = 23.5^k$

8. Bolt Shear
4 7/8 in. dia. A325-N bolts, consider eccentricity

Using LRFD Table X and the following values:

b = 3 in.
$x_o = 4 - 1.5 = 2.5$ in. ∴ C = 1.03 (effective number of bolts)
n = 2

$\phi R_b = \phi r_vA_bC = 0.75(48)(0.6013)(1.03) = 22.3^k$

9. Beam Web Bearing/Tear-out
e = 1.5 in. > $1\ 1/2d_b$ & s = 3 in. > $3d_b$ ∴ Only bearing applies

$\phi R_n = \phi\sum_{bolts} 2.4F_ud_bt = 2 \times 0.75(2.4)(58)(7/8)(0.295) = 53.9^k$ (for B5)

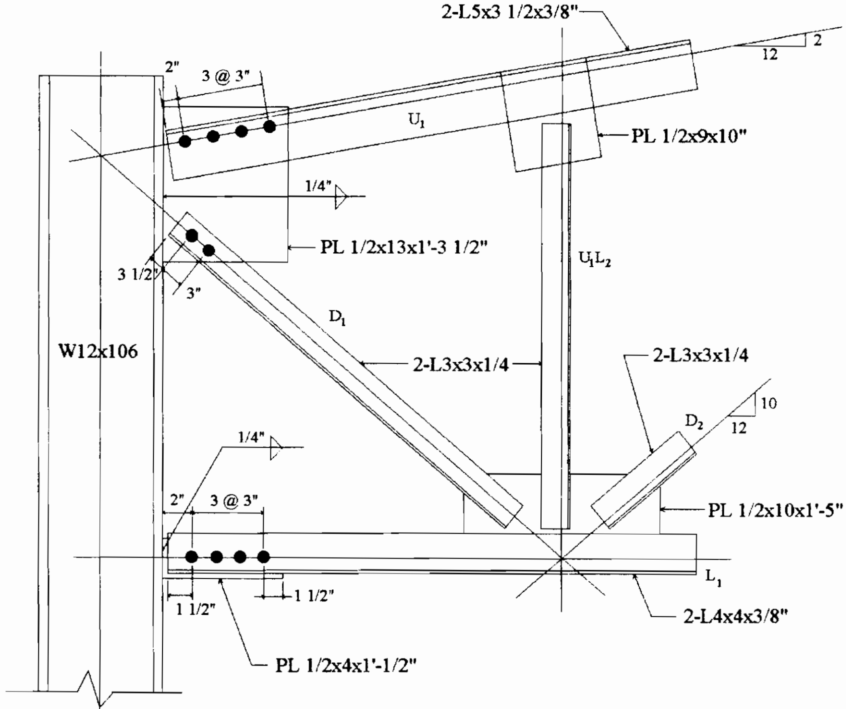
$\phi R_n = \phi\sum_{bolts} 2.4F_ud_bt = 2 \times 0.75(2.4)(58)(7/8)(0.275) = 50.2^k$ (for B8)

∴ Bolt Shear controls the connection design ⇒ $\phi V_n = 22.3^k$

3.4.5 Built-Up Truss Connection

Example 1. **Built-Up Truss** - Refer to truss B7 connected to column C2. (Fig. 3.5, 3.6)

Given: A built-up truss connected to a W12x106 column.
7/8" ϕ A325-n bolts.
All Steel is A36.
See Fig. 3.31 below.



Note: All angles welded to gusset plates use 1/4" fillets

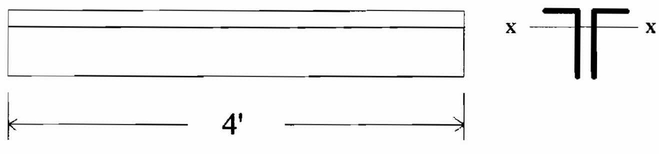
Figure 3.31

Show: All limit states that apply.

Limit States:

For Top Chord (U_1): 2-L5x3 1/2x3/8

1. Compressive Capacity



a) Buckling of Single Angle:

$$\left.\frac{kL}{r}\right)_z = \frac{4(12)}{0.762} = 63 \text{ (for single)}$$

$$\left.\frac{kL}{r}\right)_x = \frac{4(12)}{1.60} = 30 < 63 \text{ (for double)}$$

$$\left.\frac{kL}{r}\right)_y = \frac{4(12)}{1.46} = 32.9 < 63 \text{ (for double)}$$

Since $x_o = y_o$, use LRFD eq. A - E3 - 5

$$F_{exyz} = \left[\frac{\pi^2 EC_w}{(k_z L)^2} + GJ\right]\frac{1}{I_x + I_y}$$

$$F_{exyz} = \left[\frac{\pi^2 (29000)(0.217)}{(1.0(48))^2} + 11200(0.153)\right]\frac{1}{7.78 + 3.18} = 158.9 \text{ ksi}$$

b) x-x Buckling of Double Angle:

$$\left.\frac{kL}{r}\right)_x = 30 \Rightarrow \phi F_{cr} = 29.18 \text{ ksi}$$

$$\therefore \phi P_{nx} = 29.18(6.09) = 177.7^k$$

c) Flexural Torsional Buckling:

$$F_{eyz} = \frac{F_{ey} + F_{ez}}{2H}\left(1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right)$$

$$\frac{a}{r_z} = \frac{48}{0.762} = 63 > 50 \quad \therefore \text{ Need to use } \left.\frac{kL}{r}\right)_m$$

$$\left.\frac{kL}{r}\right)_m = \sqrt{\left(\left.\frac{kL}{r}\right)_y\right)^2 + \left(\frac{a}{r_z} - 50\right)^2}$$

$$\left.\frac{kL}{r}\right)_m = \sqrt{(32.9)^2 + (63 - 50)^2} = 30.22$$

$$F_{ez} = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)_m^2} = \frac{\pi^2 (29000)}{(30.22)^2} = 313.4 \text{ ksi}$$

$$F_{ez} = \left[\frac{\pi^2 E C_w}{(k_z L)^2} + GJ\right]\frac{1}{I_x + I_y}$$

$$F_{ez} = \left[\frac{\pi^2 (29000)(2x0.217)}{(4(12))^2} + 11200(2x0.153)\right]\frac{1}{15.6 + 13} = 121.7 \text{ ksi}$$

$$\therefore F_{eyz} = \frac{313.4 + 121.7}{2(0.682)}\left(1 - \sqrt{1 - \frac{4(313.4(121.7)(0.682)}{(313.4 + 121.7)^2}}\right) = 105 \text{ ksi}$$

$$F_e = \min \left| F_{exyz} \text{ or } F_{eyz} \right. = 105 \text{ ksi}$$

$$\lambda_e = \sqrt{\frac{36}{105}} = 0.60 < 1.5$$

$$\therefore \ \phi F_{cr} = 0.85(0.658^{(0.6)^2})36 = 26.5 \text{ ksi}$$

$$\phi P_n = 26.5(6.09) = 161.4^k$$

$$\text{Compressive Strength} = \min \left| \begin{array}{l} 177.7 \\ 161.4 \end{array} \right.$$

$$\therefore \ \phi P_n = 161.4^k$$

2. Angle Bearing / Tear-out
No edge distance concern and s > $3d_b$ $\therefore$ Only bearing applies.

$$\phi R_n = 4(0.75)(2.4 \text{ x } 58)(7/8)(3/8)2 = 274^k$$

3. Bolt Shear

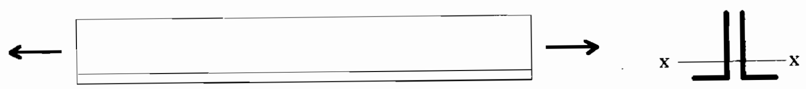
$$\phi R_n = (0.75)(48)(0.6013)4 \text{ x } 2 = 173.2^k$$

4. Weld Rupture at Gusset Plate (L_{weld} = 9 - 2(1/4) = 8.5 in.)

$$\phi R_n = 2 \text{ x } 4(1.392)(8.5) = 94.7^k$$

For Diagonal Chord (D_1): 2-L3x3x1/4

5. Tension Yield



$\phi R_n = 0.9(36)(2.88) = 93.3^k$

6. Tension Rupture

$\phi R_n = \phi F_u A_e \; , \; A_e = UA_n$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \left(\frac{0.842}{3}\right) = 0.72$$

$A_e = 0.72[2.88 - 2(7/8 + 1/8)0.25] = 1.71 \text{ in}^2$

$\therefore \; \phi R_n = 0.75(58)(1.71) = 74.5^k$

7. Angle Bearing / Tear-out
$e > 1.5d_b$ and $s > 3d_b$ $\therefore$ Only bearing applies

$\phi R_n = 0.75(2.4 \text{ x } 58)(7/8)(0.25)4 = 121.8^k$

8. Bolt Shear

$\phi R_n = 2(0.75)(48)(0.6013)2 = 86.6^k$

9. Weld Rupture at Gusset Plate (use 2 1/4" fillets, 4 inches long)

$\phi R_n = 2 \text{ x } 2 \text{ x } (4)(1.392)(4) = 89.1^k$

10. Gusset Plate Yield at Weld

$\phi R_n = 0.9(0.6 \text{ x } 36)(4 \text{ x } 4)(0.5) = 155.5^k$

For Diagonal Chord (D_2):

Assume same limit states as for Diagonal D_1.

For Bottom Chord (L_1): 2-L4x4x3/8

11. Tension Yield

$\phi R_n = 0.9(36)(5.72) = 185.3^k$

12. Tension Rupture

$\phi R_n = \phi F_u A_e \; , \; A_e = UA_n$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \left(\frac{1.14}{9}\right) = 0.87$$

$$A_e = 0.87[5.72 - 2(7/8 + 1/8)0.375] = 4.32 \text{ in}^2$$

$$\therefore \ \phi R_n = 0.75(58)(4.32) = 188.1^k$$

13. Angle Bearing / Tear-out

$e > 1.5d_b$ and $s > 3d_b$ $\therefore$ Only bearing applies

$$\phi R_n = 0.75(2.4 \text{ x } 58)(7/8)(0.375)8 = 365.4^k$$

14. Bolt Shear

$$\phi R_n = 4(0.75)(48)(0.6013)2 = 173.2^k$$

15. Weld Rupture at Gusset Plate (use 2 1/4" fillets, L_{weld} = 17 - 2(.25) = 16.5 in.)

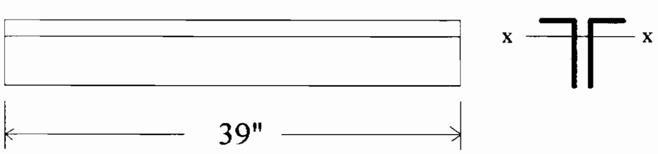
$$\phi R_n = 2 \text{ x } (4)(1.392)(16.5) = 183.7^k$$

16. Gusset Plate Yield at Weld

$$\phi R_n = 0.9(0.6 \text{ x } 36)(2 \text{ x } 16.5)(0.5) = 320.8^k$$

For Vertical Chord (U_1L_1): 2-L3x3x1/4

17. Compressive Capacity



a) Buckling of Single Angle:

$$\left.\frac{kL}{r}\right)_z = \frac{39}{0.592} = 65.9 \text{ (for single)}$$

$$\left.\frac{kL}{r}\right)_x = \frac{39}{0.93} = 41.9 < 65.9 \text{ (for double)}$$

$$\left.\frac{kL}{r}\right)_y = \frac{39}{1.39} = 28.1 < 65.9 \text{ (for double)}$$

$$F_{exyz} = \left[\frac{\pi^2 E C_w}{(k_z L)^2} + GJ\right]\frac{1}{I_x + I_y}$$

$$F_{exyz} = \left[\frac{\pi^2(29000)(0.0206)}{(1.0(39))^2} + 11200(0.0322)\right]\frac{1}{1.24 + 1.24} = 147 \text{ ksi}$$

b) x-x Buckling of Double Angle:

$$\left.\frac{kL}{r}\right)_x = 41.9 \Rightarrow \phi F_{cr} = 27.9 \text{ ksi}$$

$$\therefore \phi P_{nx} = 27.9(2.88) = 80.4^k$$

c) Flexural Torsional Buckling

$$F_{eyz} = \frac{F_{ey} + F_{ez}}{2H}\left(1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{\left(F_{ey} + F_{ez}\right)^2}}\right)$$

$$\frac{a}{r_z} = \frac{39}{0.592} = 65.9 > 50 \quad \therefore \text{Need to use} \left.\frac{kL}{r}\right)_m$$

$$\left.\frac{kL}{r}\right)_m = \sqrt{\left(\left.\frac{kL}{r}\right)_y\right)^2 + \left(\frac{a}{r_z} - 50\right)^2}$$

$$\left.\frac{kL}{r}\right)_m = \sqrt{(28.1)^2 + (65.9 - 50)^2} = 23.2$$

$$F_{ey} = \frac{\pi^2 E}{\left.\frac{kL}{r}\right)_m^2} = \frac{\pi^2(29000)}{(23.2)^2} = 533.2 \text{ ksi}$$

$$F_{ez} = \left[\frac{\pi^2 E C_w}{(k_z L)^2} + GJ\right]\frac{1}{I_x + I_y}$$

$$F_{ez} = \left[\frac{\pi^2(29000)(2x0.0206)}{(39)^2} + 11200(2x0.0322)\right]\frac{1}{2.49 + 1.39} = 187.9 \text{ ksi}$$

$$\therefore F_{eyz} = \frac{533.2 + 187.9}{2(0.844)}\left(1 - \sqrt{1 - \frac{4(533.2)(187.9)(0.844)}{(533.2 + 187.9)^2}}\right) = 174.6 \text{ ksi}$$

$$\lambda_e = \sqrt{\frac{36}{147}} = 0.5 < 1.5$$

$$\therefore \phi F_{cr} = 0.85(0.658^{(0.5)^2})36 = 27.6 \text{ ksi}$$

$$\phi P_n = 27.6(2.88) = 79.5^k$$

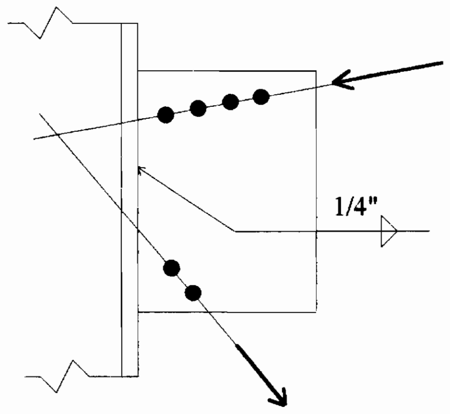
18. Weld Rupture at Gusset Plate (use 2 1/4" fillets, 2.5 and 5.5 inches long)

$$\phi R_n = 2 \times 2 \times (4)(1.392)(2.5 + 5.5)(0.5) = 89.1^k$$

19. Gusset Plate Yield at Weld

$$\phi R_n = 2 \times 0.9(0.6 \times 36)(2.5 + 5.5)(0.5) = 155.5^k$$

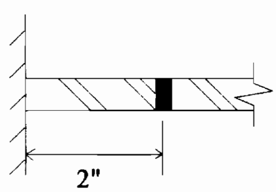
For Gusset Plate Connection at Column (U_1):



20. Plate Bearing / Tear-out (for top bolts)
No edge concern, $s > 3d_b$

$$\therefore \ \phi R_n = 0.75(2.4 \times 58)(7/8)(0.5)4 = 182.7^k$$

21. Compressive Capacity of Plate



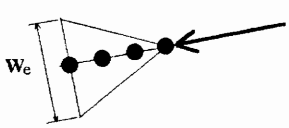
use k = 0.8 and b = 10.4 in.

$$I_x = \frac{1}{12}(10.4)(0.5)^3 = 0.108 \text{ in}^4$$

$$A_g = 10.4(0.5) = 5.2 \text{ in}^2$$

$$r_x = \sqrt{\frac{0.108}{5.2}} = 0.144 \text{ in.}$$

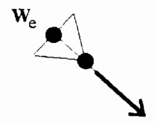
$$\frac{kL}{r} = \frac{0.8(2)}{0.144} = 11.1 \Rightarrow \phi F_{cr} = 30.4 \text{ ksi}$$



w_e = 10.4 in. (Whitmore Model)

$$\phi R_n = 30.4(5.2) = 158.1^k$$

22. Plate Tension Yield (bottom bolts)



$\phi R_n = 0.9(36)(3.5 \times 0.5) = 56.7^k$

w_e = 3.5 in. (Whitmore Model)

23. Plate Tension Rupture

$\phi R_n = 0.75(58)[3.5 - 1(7/8 + 1/8)](0.5) = 54.4^k$

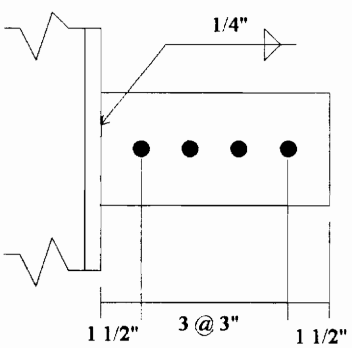
24. Weld Rupture

$\phi R_n = 4(1.392)(15.5)\text{x}2 = 172.6^k$

25. Column Flange Yield at Weld

$\phi R_n = 0.9(0.6 \times 36)(15.5)(0.99)\text{x}2 = 596.6^k$

For Gusset Plate Connection to Column (L_1):



26. Plate Tension Yield

$\phi R_n = 0.9(36)(4 \times 0.5) = 64.8^k$

27. Plate Tension Rupture

$\phi R_n = 0.75(58)[4 - 1(7/8 + 1/8)](0.5) = 65.3^k$

28. Plate Bearing / Tear-out
edge > d_b and spacing > $3d_b$

$\therefore\ \phi R_n = 0.75(2.4 \times 58)(7/8)(0.5)4 = 182.7^k$

29. Weld Rupture

$\phi R_n = 4(1.392)(4)x2 = 44.5^k$

30. Column Flange Yield at Weld

$\phi R_n = 0.9(0.6 \times 36)(4)(0.99)x2 = 154^k$

Column Side:

31. Local Web Yielding

$\phi R_n = \phi(N + 5k)t_w F_y = 1.0[15.5 + 5(1.5875)](0.61)(36) = 525.7^k$

32. Web Buckling

$$\phi R_n = \phi \frac{4100 t_{wc}^3 \sqrt{F_{yc}}}{d_c} = 0.9 \frac{4100(0.61)^3 \sqrt{36}}{9.5} = 529^k$$

33. Local Web Crippling

$$\phi R_n = \phi 135 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\phi R_n = 0.85(135)(0.61)^2 \left[1 + 3\left(\frac{15.5}{12.89}\right)\left(\frac{0.61}{0.99}\right)^{1.5}\right]\sqrt{\frac{36(0.99)}{0.61}} = 895.8^k$$

Assuming $\phi R_n > F_{fu}$, no stiffeners are required for C2 at truss connection.

3.4.6 Hanger Plate Connection

Example 1. **Beam Hanger Plate** - Refer to beam B6 hanger plate and threaded rod connection. (Fig. 3.3, 3.4, 3.5)

Given: A W16x40 beam is supported by a plate and threaded rod
PL 3/4x4x1'-0"
1 3/8" ϕ Threaded Rod (with #3 clevis and 1 1/2" ϕ pin)
All Steel is A36.
See Fig. 3.32 below.

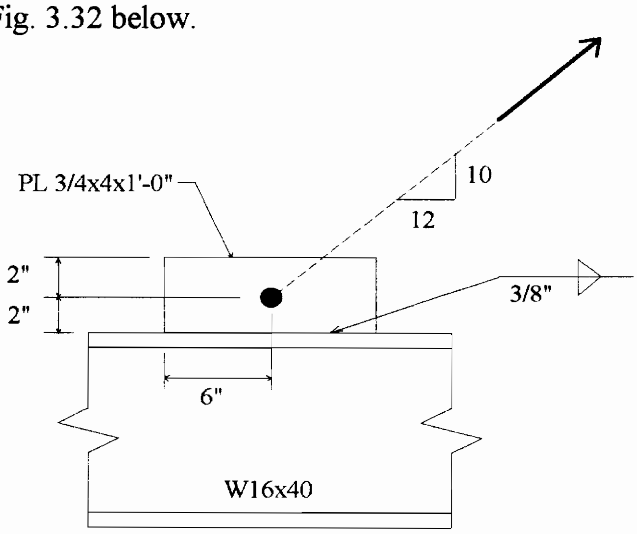


Figure 3.32

Show: All limit states that apply.

Limit States:

1. Weld Rupture

   Using LRFD Table XVIII and the values:

   l = 12 in.
   al = 2 in. $\Rightarrow$ a = 0.167
   kl = 0 $\Rightarrow$ k = 0

   Interpolating gives, C = 2.39

   $\therefore \ \phi R_n = 1.0(2.39)(6)(12) = 172.1^k$

2. Beam Flange Yield at Weld

   $\phi R_n = 0.9(0.6 \times 36)(12 \times 0.505) = 117.8^k$

3. Plate Yield at Weld

$\phi R_n = 0.9(0.6 \text{ x } 36)(12 \text{ x } 0.75) = 175^k$

4. Plate Tension Yield

$\phi R_n = 0.9(36)[2 \text{ x } 2 / \cos 39.8°](0.75) = 126.5^k$

5. Plate Tension Rupture

$\phi R_n = 0.75(58)[(2 \text{ x } 2 / \cos 39.8°) - (1\ 1/2 + 1/8)](0.75) = 116.8^k$

6. Plate Bearing / Tear-out

e > $1.5d_b$ and s > $3d_b$ ∴ Only bearing applies.

$\phi R_n = 0.75(2.4 \text{ x } 58)(1\ 1/2 + 1/8)(0.75) = 127.2^{k-}$

7. Tensile Capacity of Rod

$\phi T_n = 0.75F_u(0.75A_g) = 0.75(58)(0.75)[\pi(1.375)^2 / 4] = 48.4^k$

∴ Tensile Capacity of Threaded Rod controls the connection ⇒ $\phi T_n = 48.4^k$

3.4.7 STD Pipe Column Design

Example 1. **Column Design** - Refer to column C3 (Fig. 3.3, 3.4)

Given: A 4" ϕ STD Pipe Column between B4 and B8.
Steel is A36.
See Fig. 3.33 below.

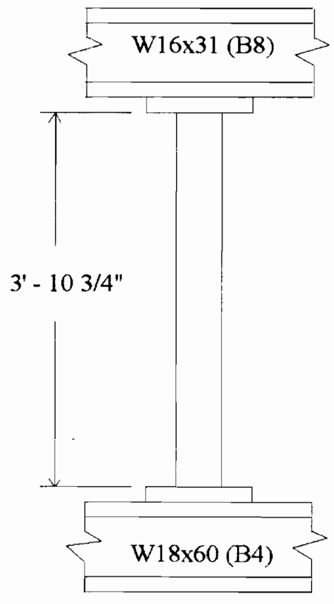


Figure 3.33

Show: All limit states that apply.

Limit States:

1. Local Buckling
Using LRFD Table B5.1

$$\frac{D}{t} = \frac{4}{0.237} = 16.9 \leq \frac{3300}{F_y} = 91.7 \quad \therefore \text{ ok}$$

2. Compressive Capacity

Using LRFD Column Load Tables, k = 1.0

kL = 1.0(3.9) = 3.9 ft.

Interpolation gives, $\phi P_n = 90^k$

CHAPTER IV
CONCLUSION

This report has looked at a number of different steel connections which are used in industry today, focusing mainly on simple framing and moment connections. All the necessary design limit states have been given to the reader from both the AISC LRFD and ASD Specifications. Over twenty different building connection design examples have been covered to give students a realistic idea of how they are connected and what limit states control their design. This report's purpose is to be used as a teaching aid for structural steel design in order to familiarize students with connection design and the associated strength limit states.

Wherever possible, the report included print-outs from an AISC computer software package, CONXPRT, in both LRFD and ASD. Studying both design methods will allow students to compare results between the two techniques.

Finally, the building connections have been linked to a structural steel "sculpture" to reinforce the students' understanding of the basic building connections through visualization. Fabrication and erection drawings are provided for this "sculpture."

It is the author's desire that this report helps students visualize and understand the connections presented, and that understanding will carry over to other areas in structural steel connection design.

REFERENCES

AISC (1986). *Manual for Steel Construction, Load and Resistance Factor Design,* 1st ed., American Institute of Steel Construction, Chicago, IL.

AISC (1989). *Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design,* 9th ed., American Institute of Steel Construction, Chicago, IL.

AISC (1992). *Manual for Steel Construction - Connections (ASD),* 9th ed., Vol. II, American Institute of Steel Construction, Chicago, IL.

AISC (1992a). *Manual for Steel Construction - Connections (LRFD),* 1st ed., Vol. II, American Institute of Steel Construction, Chicago, IL.

AISC (1993). *Specification for Structural Steel Buildings -- Load and Resistance Factor Design,* Proposed 2nd ed., American Institute of Steel Construction, Chicago, IL.

Blodgett, O. W. (1966). *Design of Welded Structures*, James F. Lincoln Arc Welding Foundation, Cleveland, OH, Section 3.3.

CONXPRT (1992). *User's Manual for CONXPRT Module I - ASD Shear Connections*, Version 1.1, American Institute of Steel Construction, Chicago, IL.

CONXPRT (1992a). *User's Manual for CONXPRT Module II - ASD Moment Connections*, Version 1.0, American Institute of Steel Construction, Chicago, IL.

CONXPRT (1992b). *User's Manual for CONXPRT FOR THE CLASSROOM Module I - LRFD Framing Connections*, Version 1.0, American Institute of Steel Construction, Chicago, IL.

DeWolf, J. T. and Ricker, D. T. (1991). *Column Base Plates, Steel Design Guide Series,* 2nd ed., American Institute of Steel Construction, Chicago, IL.

Driscoll, G. C. and Beedle, L. S. (1982). "Suggestions for Avoiding Beam-to-Column Web Connection Failure," *Engineering Journal*, AISC, 19(1), 16-19.

Easterling, W. S. (1992). *Class Notes: Design of Steel Structures*, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Garrett, J. H. and Brockenbrough, R. L. (1986). "Design Loads for Seated-beam in LRFD," *Engineering Journal*, AISC, 23(2), 84-88.

Murray, T. M. (1990). *Extended End-Plate Moment Connections, Steel Design Guide Series,* American Institute of Steel Construction, Chicago, IL.

Murray, T. M. (1993). *Class Notes: Advanced Steel Design*, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Rentchler, G. P., Chen, W. F. and Driscoll, G. C. (1982). "Beam-to-Column Web Connections Details," *Journal of the Structural Division*, ASCE, 108(ST2), February, 393-409.

Salmon, C. G. and Johnson, V. E. (1990). *Steel Structures - Design and Behavior,* 3rd ed., Harper and Row, New York.

Whitmore, R. E. (1952). "Experimental Investigation of Stresses in Gusset Plates," *Bulletin No. 16*, Engineering Experiment Station, University of Tennessee, Knoxville.

APPENDIX A

CONXPRT OUTPUT (LRFD)

CONXPRT LRFD for the Classroom Version 1.0 from AISC, Chicago, IL.
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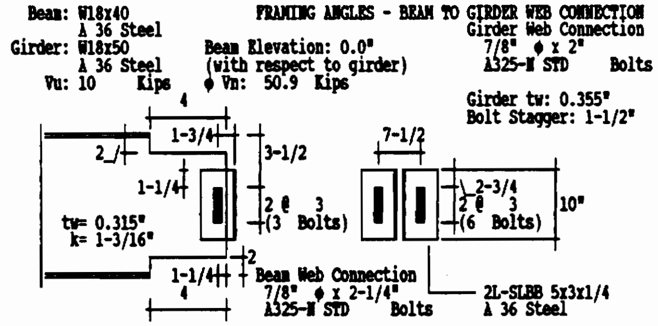
Job Id: M.E.C.E. PROJECT
Con. Id: B1A TO B1



__Connection Description__:

Framing angles, beam to girder web connection.

Beam: W18x40 A 36 Steel F_y= 36 ksi F_u = 58 ksi
d= 17.9 in. t_w = 0.315 in.

Girder: W18x50 A 36 Steel F_y= 36 ksi F_u= 58 ksi
t_w = 0.355 in.

Factored beam end reaction, V_u: 10 Kips
Connection strength, ϕ V_n: 50.9 Kips

__Strength Limit States__:

Beam gross shear strength (AISC LRFD Sect. F2):
$\phi V_n = (0.9) \times (0.6 F_y A_w)$ = 85.1 Kips ≥ 10 Kips OK

Beam net shear strength (AISC LRFD Sect. J4):
$\phi V_n = (0.75) \times (0.6 F_u A_n)$ = 91.2 Kips ≥ 10 Kips OK

Beam web bearing/tearout strength (AISC LRFD Sect. J3.6, J3.9 & J3.10):
$\phi V_n = (0.75) V_n$ = 86.3 Kips ≥ 10 Kips OK

__Beam web connection__:

Use three (3) 7/8" ϕ x 2-1/4" w/ 5/32" thick washers
A325-N STD bolts in double shear
(User may need to adjust bolt length for other conditions.)
Total bolt shear design strength:
ϕR_b = 126.6 Kips ≥ 10 Kips OK (AISC LRFD J3.3)

Block shear strength of coped beam web (AISC LRFD Sect. J5.2.c):
ϕV_{bs} = 0.75 V_{bs} = 50.9 Kips ≥ 10 Kips OK

Bending strength of coped beam web:
(Design and Behavior of Coped Beams, University of Texas at Austin, July 1984)
ϕV_{cb} = 0.9 V_{cb} = 73 Kips ≥ 10 Kips OK (Bending controls)

Girder web connection:

Use six (6) 7/8 in. ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
(User may need to adjust bolt length for other conditions.)
Total bolt shear design strength:
ϕR_b = 126.6 Kips ≥ 10 Kips OK (AISC LRFD J3.3)

Girder web bearing/tearout strength (AISC LRFD Sect. J3.6):
ϕP_b = 0.75 P_b = 194.6 Kips ≥ 10 Kips OK
Remaining web bearing/tearout strength= 184.6 Kips

Framing angles:

2L-SLBB 5x3x1/4 x 10"
A 36 Steel Angles F_y = 36 ksi F_u = 58 ksi

Angle gross shear strength (AISC LRFD Equ. J5-3):
ϕV_n = 0.8 V_n = 100.8 Kips ≥ 10 Kips OK

Beam side angle leg design strengths:
Net shear (AISC LRFD Sect. J4):
ϕV_n = 0.75 V_n = 93.8 Kips ≥ 10 Kips OK
Block shear (AISC LRFD Sect. J5.2.c):
ϕV_{bs} = 0.75 V_{bs} = 81 Kips ≥ 10 Kips OK
Bearing/Tearout (AISC LRFD Sect. J3.6, J3.9 & J3.10):
ϕP_b = 0.75 P_b = 118.5 Kips ≥ 10 Kips OK

Girder side angle leg design strengths:
Net shear (AISC LRFD Sect. J4):
ϕV_n = 0.75 V_n = 93.8 Kips ≥ 10 Kips OK
Block shear (AISC LRFD Sect. J5.2.c):
ϕV_{bs} = 0.75 V_{bs} = 102.6 Kips ≥ 10 Kips OK
Bearing/Tearout (AISC LRFD Sect. J3.6, J3.9 & J3.10):
ϕP_b = 0.75 P_b = 137 Kips ≥ 10 Kips OK

-- End --

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Con. Id: B2A TO B2

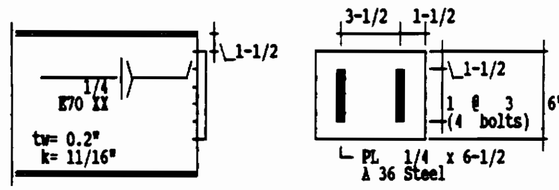
SHEAR END PLATE - BEAM TO GIRDER WEB CONNECTION

Beam: W12x14
A 36 Steel
Girder: W18x60
A 36 Steel
Vu: 10 Kips

Beam Elevation: -3"
(with repsect to girder)
ϕ Vn: 22.2 Kips

Girder Web Connection
7/8" ϕ x 2"
A325-N STD Bolts

Girder tw: 0.415"



Connection Description:

Shear end plate, beam to girder web connection.

Beam: W12x14 A 36 Steel F_y= 36 ksi F_u= 58 ksi
d= 11.91 in. t_w= 0.2 in.

Girder: W18x60 A 36 Steel F_y= 36 ksi F_u= 58 ksi
t_w = 0.415 in.

Factored beam end reaction, Vu: 10 Kips
Connection strength, ϕ Vn: 22.2 Kips

Strength Limit States:

Beam gross shear strength (AISC LRFD Sect. F2):
$\phi V_n = (0.9) \times (0.6 F_y A_w) = 46.3$ Kips $\geq$ 10 Kips OK

Maximum beam web strength at plate= $0.8 \times (0.7 F_y)(L_p - 2x t_{weld}) \times t_w$
= 22.2 Kips $\geq$ 10 Kips OK

Beam web connection:
Use two 1/4 in. - E70 XX fillet welds
Total weld shear strength = 61.2 Kips $\geq$ 10 Kips OK

Girder web connection:

Use four (4) 7/8 in. ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
(User may need to adjust bolt length for other conditions.)
Total bolt shear design strength:
$\phi\ R_b$= 84.4 Kips $\geq$ 10 Kips OK (AISC LRFD J3.3)

Girder web bearing/tearout strength (AISC LRFD Sect. J3.6):
$\phi\ P_b$= 0.75 P_b= 151.6 Kips $\geq$ 10 Kips OK
Remaining web bearing/tearout strength= 141.6 Kips

<u>Connection Plate</u>:

PL 1/4 x 6-1/2 x 6"
A 36 Steel plate F_y= 36 ksi F_u= 58 ksi

Plate gross shear strength (AISC LRFD Equ. J5-3):
$\phi\ V_n$= 0.8 V_n= 60.5 Kips $\geq$ 10 Kips OK
Plate net shear strength (AISC LRFD Sect. J4):
$\phi\ V_n$= 0.75 V_n= 53.8 Kips $\geq$ 10 Kips OK
Plate block shear strength (AISC LRFD Sect. J5.2.c):
$\phi\ V_{bs}$= 0.75 V_{bs}= 60.6 Kips $\geq$ 10 Kips OK
Plate bearing/tearout strength (AISC LRFD Sect. J3.6, J3.9 & J3.10):
$\phi\ P_b$= 0.75 P_b= 91.3 Kips $\geq$ 10 Kips OK

-- End --

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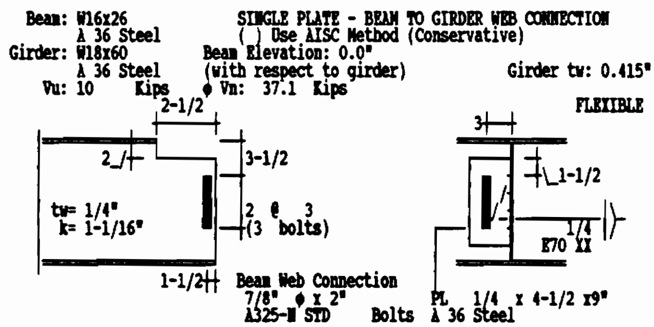
Con. Id: B2B TO B2



<u>Connection Description</u>:

Single plate (shear tab), beam to girder web connection.
Note:
The girder web is treated as as rotationally flexible element.

Beam: W16x26 A 36 Steel F_y= 36 ksi F_u= 58 ksi
d= 15.69 in. t_w= 0.25 in.

Girder: W18x60 A 36 Steel F_y= 36 ksi F_u= 58 ksi
t_w = 0.415 in.

Factored beam end reaction, V_u: 10 Kips
Connection strength, ϕ V_n: 37.1 Kips

<u>Strength Limit States</u>:

Beam gross shear strength (AISC LRFD Sect. F2):
$\phi V_n = (0.9) \times (0.6\ F_y\ A_w)$= 66.5 Kips ≥ 10 Kips OK

Beam net shear strength (AISC LRFD Sect. J4):
$\phi V_n = 0.75\ V_n$= 71 Kips ≥ 10 Kips OK

Beam web bearing/tearout strength (AISC LRFD Sect. J3.6, J3.9 & J3.10):
(Effective number of bolts= C= 1.76)
$\phi P_b = 0.75\ P_b$= 40.2 Kips ≥ 10 Kips OK

<u>Beam web connection</u>:

Use three (3) 7/8 in. ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
(User may need to adjust bolt length for other conditions.)
Total bolt shear design strength:
$\phi\ r_v$= 21.11 Kips / bolt (AISC LRFD J3.3)
C= 1.76
the value of C was obtained from Table X in Part 5 of
the AISC LRFD Manual using:
b= 3 in. X_o= 3 in. n= 3
$\phi\ R_b$= C x ($\phi\ r_v$)
= 37.1 Kips ≥ 10 Kips OK

Block shear strength of coped beam web (AISC LRFD Sect. J5.2.c):
$\phi\ V_{bs}$= 0.75 V_{bs}= 42.1 Kips ≥ 10 Kips OK

Bending strength of coped beam web:
(Engineering for Steel Construction, Appendix B)
$\phi\ V_{cb}$= 0.9 V_{cb}= 94 Kips ≥ 10 Kips OK (Bending controls)

<u>Connection Plate</u>:

PL 1/4 x 4-1/2 x 9"
A 36 Steel plate F_y= 36 ksi F_u= 58 ksi

Plate gross shear strength (AISC-LRFD Equ. J5-3):
$\phi\ V_n$= 0.8 V_n= 45.4 Kips ≥ 10 Kips OK
Plate net shear strength (AISC LRFD Sect. J4):
$\phi\ V_n$= 0.75 V_n= 40.4 Kips ≥ 10 Kips OK
Plate block shear strength (AISC LRFD Sect. J5.2.c):
$\phi\ V_{bs}$= 0.75 V_{bs}= 43.8 Kips ≥ 10 Kips OK
Plate Bearing/tearout strength (AISC LRFD Sect. J3.6, J3.9 & J3.10):
(Effective number of bolts= C= 1.76)
$\phi\ P_b$= 0.75 P_b= 40.2 Kips ≥ 10 Kips OK

<u>Girder web connection</u>:

Connection welds are designed for the gross shear strength
of the plate (45.4 Kips).

Use two 1/4" - E70 XX fillet welds
e_w= 3 in. C= 1.766
The value of C was obtained from Table XVIII in Part 5 of
the AISC LRFD Manual using: a= 0.33 k= 0.0

Maximum reaction (based on weld strength):
ϕR_n= 63.6 Kips ≥ 45.4 Kips OK

Girder web shear strength at weld= 150.6 Kips ≥ 10 Kips OK
Remaining girder web shear strength= 140.6 Kips

-- End --

- **Note:**
  Number of bolt rows has been increased to satisfy minimum plate length requirements.

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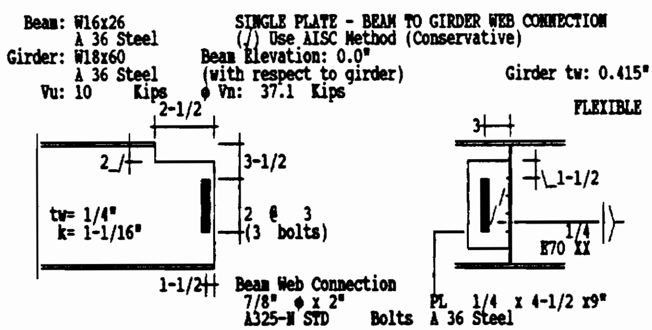

__Connection Description__:

Single plate (shear tab), beam to girder web connection.
Note:
The girder web is treated as as rotationally flexible element.

The method described in the 9th Edition of the AISC-ASD Manual was used to design this connection, except that Load and Resistance Factors were used instead of Safety Factors. The AISC-ASD method is conservative.

Beam: W16x26 A 36 Steel F_y= 36 ksi F_u= 58 ksi
d= 15.69 in. t_w= 0.25 in.

Girder: W18x60 A 36 Steel F_y= 36 ksi F_u= 58 ksi
t_w = 0.415 in.

Factored beam end reaction, V_u: 10 Kips
Connection strength, $\phi\ V_n$: 37.1 Kips

__Strength Limit States__:

Beam gross shear strength (AISC LRFD Sect. F2):
$\phi\ V_n$= (0.9) x (0.6 F_y A_w)= 66.5 Kips ≥ 10 Kips **OK**

Beam net shear strength (AISC LRFD Sect. J4):
$\phi\ V_n$= 0.75 V_n= 71 Kips ≥ 10 Kips **OK**

Beam web bearing/tearout strength (AISC LRFD Sect. J3.6, J3.9 & J3.10):
(Effective number of bolts= C= 1.76)
ϕP_b= 0.75 P_b= 40.2 Kips ≥ 10 Kips OK

Beam web connection:

Use three (3) 7/8 in. ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
(User may need to adjust bolt length for other conditions.)
Total bolt shear design strength:
ϕr_v= 21.11 Kips / bolt (AISC LRFD J3.3)
C= 1.76
the value of C was obtained from Table X in Part 5 of
the AISC LRFD Manual using:
b= 3 in. X_o= 3 in. n= 3
ϕR_b= C x (ϕr_v)
= 37.1 Kips ≥ 10 Kips OK

Block shear strength of coped beam web (AISC LRFD Sect. J5.2.c):
ϕV_{bs}= 0.75 V_{bs}= 42.1 Kips ≥ 10 Kips OK

Bending strength of coped beam web:
(Engineering for Steel Construction, Appendix B)
ϕV_{cb}= 0.9 V_{cb}= 94 Kips ≥ 10 Kips OK (Bending controls)

Connection Plate:

PL 1/4 x 4-1/2 x 9"
A 36 Steel plate F_y= 36 ksi F_u= 58 ksi

Plate gross shear strength (AISC-LRFD Equ. J5-3):
ϕV_n= 0.8 V_n= 45.4 Kips ≥ 10 Kips OK
Plate net shear strength (AISC LRFD Sect. J4):
ϕV_n= 0.75 V_n= 40.4 Kips ≥ 10 Kips OK
Plate block shear strength (AISC LRFD Sect. J5.2.c):
ϕV_{bs}= 0.75 V_{bs}= 43.8 Kips ≥ 10 Kips OK
Plate bearing/tearout strength (AISC LRFD Sect. J3.6, J3.9 & J3.10):
(Effective number of bolts= C= 1.76)
ϕP_b= 0.75 P_b= 40.2 Kips ≥ 10 Kips OK

Girder web connection:

Connection welds are designed for the gross shear strength
of the plate (45.4 Kips).

Use two 1/4" - E70 XX fillet welds
e_w= 3 in. C= 1.766
The value of C was obtained from Table XVIII in Part 5 of
the AISC LRFD Manual using: a= 0.33 k= 0.0

Maximum reaction (based on weld strength):
ϕR_n= 63.6 Kips ≥ 45.4 Kips OK

Girder web shear strength at weld= 150.6 Kips ≥ 10 Kips OK
Remaining girder web shear strength= 140.6 Kips

-- End --

- **Note:**
  Number of bolt rows has been increased to satisfy minimum plate length requirements.

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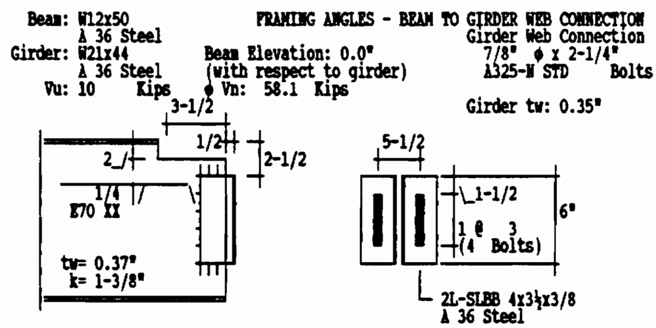
Con. Id: B3A TO B3



Connection Description:

Framing angles, beam to girder web connection.

Beam: W12x50 A 36 Steel F_y= 36 ksi F_u= 58 ksi
d= 12.19 in. t_w= 0.37 in.

Girder: W21x44 A 36 Steel F_y= 36 ksi F_u= 58 ksi
t_w = 0.35 in.

Factored beam end reaction, V_u: 10 Kips
Connection strength, ϕ V_n: 58.1 Kips

Strength Limit States:

Beam gross shear strength (AISC LRFD Sect. F2):
ϕV_n= 0.9 x (0.6 F_y A_w)= 73.3 Kips ≥ 10 Kips OK

Beam web connection:

Use 1/4 in. - E70 XX fillet welds
Total weld shear strength = 87.3 Kips ≥ 10 Kips OK
Beam web strength at weld = 58.5 Kips ≥ 10 Kips OK
Block shear strength of coped beam web:
(Engineering for Steel Construction, 1984, Page 3-33 except
that Resistance Factors are used instead of Safety Factors)
ϕV_{bs}= 0.75 V_{bs}= 58.1 Kips ≥ 10 Kips OK

Bending strength of coped beam web:
(Engineering for Steel Construction, Appendix B)
$\phi\ V_{cb}$= 0.9 V_{cb}= 84.6 Kips ≥ 10 Kips OK (Bending controls)

Girder web connection:

Use four (4) 7/8 in. ϕ x 2-1/4" w/ 5/32" thick washers
A325-N STD bolts in single shear
(User may need to adjust bolt length for other conditions.)
Total bolt shear design strength:
$\phi\ R_b$ = 84.4 Kips ≥ 10 Kips OK (AISC LRFD J3.3)

Girder web bearing/tearout strength (AISC LRFD Sect. J3.6):
$\phi\ P_b$= 0.75 P_b= 127.9 Kips ≥ 10 Kips OK
Remaining web bearing/tearout strength= 117.9 Kips

Framing angles:

2L-SLBB 4x3½x3/8 x 6"
A 36 Steel Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear strength (AISC LRFD Equ. J5-3):
$\phi\ V_n$= 0.8 V_n = 90.7 Kips ≥ 10 Kips OK

Girder side angle leg design strengths:
Net shear (AISC LRFD Sect. J4):
$\phi\ V_n$= 0.75 V_n= 80.7 Kips ≥ 10 Kips OK
Block shear (AISC LRFD Sect. J5.2.c):
$\phi\ V_{bs}$= 0.75 V_{bs}= 89.6 Kips ≥ 10 Kips OK
Bearing/Tearout (AISC LRFD Sect. J3.6, J3.9 & J3.10):
$\phi\ P_b$= 0.75 P_b= 137 Kips ≥ 10 Kips OK

-- End --

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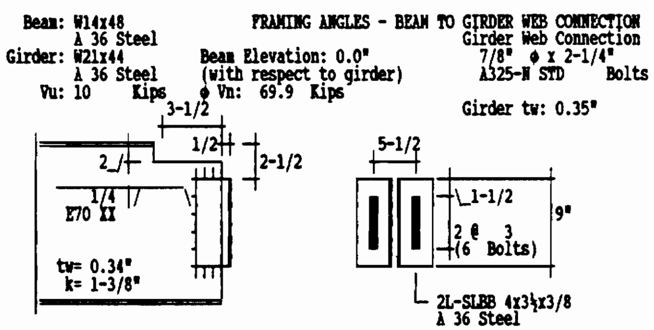
Date: 06/30/94 By: RAK Page 1
Job Id: M.E.C.E. PROJECT
Con. Id: B3B TO B3



Connection Description:

Framing angles, beam to girder web connection.

Beam: W14x48 A 36 Steel F_y= 36 ksi F_u= 58 ksi
d= 13.79 in. t_w= 0.34 in.

Girder: W21x44 A 36 Steel F_y= 36 ksi F_u= 58 ksi
t_w = 0.35 in.

Factored beam end reaction, V_u: 10 Kips
Connection strength, ϕ V_n: 69.9 Kips

Strength Limit States:

Beam gross shear strength (AISC LRFD Sect. F2):
ϕV_n= 0.9 x (0.6 F_y A_w)= 77.9 Kips ≥ 10 Kips OK

Beam web connection:

Use 1/4 in. - E70 XX fillet welds
Total weld shear strength = 126.5 Kips ≥ 10 Kips OK
Beam web strength at weld = 77.9 Kips ≥ 10 Kips OK
Block shear strength of coped beam web:
(Engineering for Steel Construction, 1984, Page 3-33 except
that Resistance Factors are used instead of Safety Factors)
ϕV_{bs}= 0.75 V_{bs}= 69.9 Kips ≥ 10 Kips OK

Bending strength of coped beam web:
(Engineering for Steel Construction, Appendix B)
$\phi V_{cb} = 0.9\ V_{cb} = 103.6$ Kips ≥ 10 Kips OK (Bending controls)

<u>Girder web connection</u>:

Use six (6) 7/8 in. ϕ x 2-1/4" w/ 5/32" thick washers
A325-N STD bolts in single shear
(User may need to adjust bolt length for other conditions.)
Total bolt shear design strength:
$\phi R_b = 126.6$ Kips ≥ 10 Kips OK (AISC LRFD J3.3)

Girder web bearing/tearout strength (AISC LRFD Sect. J3.6):
$\phi P_b = 0.75\ P_b = 191.8$ Kips ≥ 10 Kips OK
Remaining web bearing/tearout strength= 181.8 Kips

<u>Framing angles</u>:

2L-SLBB 4x3½x3/8 x 9"
A 36 Steel Angles $F_y = 36$ ksi $F_u = 58$ ksi

Angle gross shear strength (AISC LRFD Equ. J5-3):
$\phi V_n = 0.8\ V_n = 136.1$ Kips ≥ 10 Kips OK

Girder side angle leg design strengths:
Net shear (AISC LRFD Sect. J4):
$\phi V_n = 0.75\ V_n = 121.1$ Kips ≥ 10 Kips OK
Block shear (AISC LRFD Sect. J5.2.c):
$\phi V_{bs} = 0.75\ V_{bs} = 129.6$ Kips ≥ 10 Kips OK
Bearing/Tearout (AISC LRFD Sect. J3.6, J3.9 & J3.10):
$\phi P_b = 0.75\ P_b = 205.5$ Kips ≥ 10 Kips OK

-- End --

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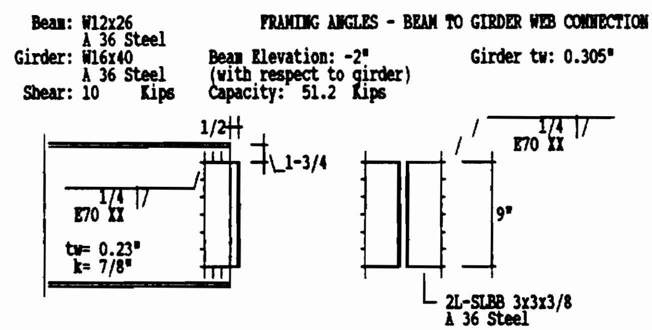
Date: 06/30/94 By: RAK Page 1
Job Id: M.E.C.E. PROJECT
Con. Id: B6A TO B6



Connection Description:

Framing angles, beam to girder web connection.

Beam: W12x26 A 36 Steel F_y= 36 ksi F_u= 58 ksi
d= 12.22 in. t_w= 0.23 in.

Girder: W16x40 A 36 Steel F_y= 36 ksi F_u= 58 ksi
t_w = 0.305 in.

Factored beam end reaction, V_u: 10 Kips
Connection strength, ϕ V_n: 51.2 Kips

Strength Limit States:

Beam gross shear strength (AISC LRFD Sect. F2):
ϕ V_n= 0.9 x (0.6 F_y A_w)= 54.6 Kips ≥ 10 Kips OK

Beam web connection:

Use 1/4 in. - E70 XX fillet welds
Total weld shear strength = 123 Kips ≥ 10 Kips OK
Beam web strength at weld = 51.2 Kips ≥ 10 Kips OK

Girder web connection:

Use 1/4 in. - E70 XX fillet welds

Total weld shear strength = 64.2 Kips ≥ 10 Kips **OK**

Girder web shear strength at weld= 110.7 Kips ≥ 10 Kips **OK**
Remaining girder web shear strength= 100.7 Kips

<u>Framing angles</u>:

2L-SLBB 3x3x3/8 x 9"
A 36 Steel Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear strength (AISC LRFD Equ. J5-3):
$\phi V_n = 0.8 V_n$ = 136.1 Kips ≥ 10 Kips **OK**

-- End --

- **Note:**
The designer should consider the erectability of this connection.

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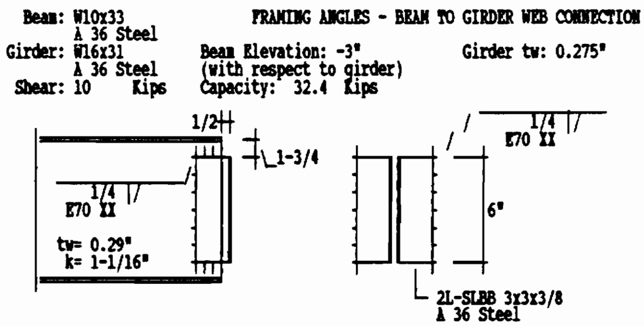
Date: 06/30/94 By: RAK Page 1
Job Id: M.E.C.E. PROJECT
Con. Id: B8A TO B8



Connection Description:

Framing angles, beam to girder web connection.

Beam: W10x33 A 36 Steel F_y = 36 ksi F_u = 58 ksi
d= 9.73 in. t_w = 0.29 in.

Girder: W16x31 A 36 Steel F_y = 36 ksi F_u = 58 ksi
t_w = 0.275 in.

Factored beam end reaction, V_u: 10 Kips
Connection strength, ϕV_n: 32.4 Kips

Strength Limit States:

Beam gross shear strength (AISC LRFD Sect. F2):
$\phi V_n = 0.9 \times (0.6 F_y A_w) = 54.9$ Kips ≥ 10 Kips OK

Beam web connection:

Use 1/4 in. - E70 XX fillet welds
Total weld shear strength = 84.2 Kips ≥ 10 Kips OK
Beam web strength at weld = 44.2 Kips ≥ 10 Kips OK

Girder web connection:

Use 1/4 in. - E70 XX fillet welds

Total weld shear strength = 32.4 Kips ≥ 10 Kips **OK**

Girder web shear strength at weld= 66.5 Kips ≥ 10 Kips **OK**
Remaining girder web shear strength= 56.5 Kips

Framing angles:

2L-SLBB 3x3x3/8 x 6"
A 36 Steel Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear strength (AISC LRFD Equ. J5-3):
$\phi V_n = 0.8 V_n = 90.7$ Kips ≥ 10 Kips **OK**

-- End --

- Note:
The designer should consider the erectability of this connection.

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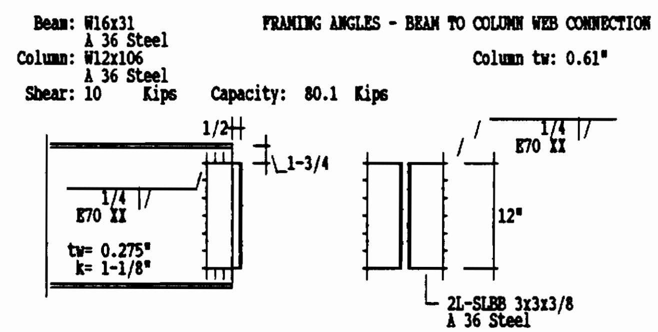
Date: 06/30/94 By: RAK Page 1
Job Id: M.E.C.E. PROJECT
Con. Id: B8 TO C2



<u>Connection Description</u>:

Framing angles, beam to column web connection.

Beam: W16x31 A 36 Steel F_y= 36 ksi F_u= 58 ksi
d= 15.88 in. t_w= 0.275 in.

Column: W12x106 A 36 Steel F_y= 36 ksi F_u= 58 ksi
t_w = 0.61 in.

Factored beam end reaction, V_u: 10 Kips
Connection strength, ϕ V_n: 80.1 Kips

<u>Strength Limit States</u>:

Beam gross shear strength (AISC LRFD Sect. F2):
ϕ V_n= 0.9 x (0.6 F_y A_w)= 84.9 Kips ≥ 10 Kips OK

<u>Beam web connection</u>:

Use 1/4 in. - E70 XX fillet welds
Total weld shear strength = 160.8 Kips ≥ 10 Kips OK
Beam web strength at weld = 80.1 Kips ≥ 10 Kips OK

<u>Column web connection</u>:

Use 1/4 in. - E70 XX fillet welds

Total weld shear strength = 99.3 Kips ≥ 10 Kips **OK**

Column web shear strength at weld= 295.1 Kips ≥ 10 Kips **OK**
Remaining column web shear strength= 285.1 Kips

Framing angles:

2L-SLBB 3x3x3/8 x 12"
A 36 Steel Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear strength (AISC LRFD Equ. J5-3):
$\phi V_n = 0.8 V_n$ = 181.4 Kips ≥ 10 Kips **OK**

-- End --

- **Note:**
The designer should consider the erectability of this connection.

APPENDIX B

CONXPRT OUTPUT
(ASD)

CONXPRT™ Module I ASD Shear Connection Design Advice
Double Framing Angles Knowledge Base
Version 2.0 from AISC, Chicago, IL.
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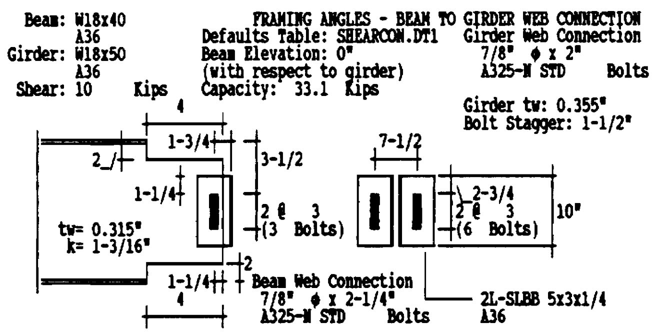
Date: 06/30/94 By: RAK Page 1
Job Id: M.E.C.E. PROJECT
Id: B1B TO B1



Connection Description:

Framing angles, beam to girder web connection.

Beam: W18x40 A36 F_y= 36.0 ksi F_u = 58.0 ksi
d= 17.9 in. t_w = 0.315 in.

Girder: W18x50 A36 F_y= 36 ksi F_u= 58 ksi
t_w = 0.355 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 33.1 Kips

Strength Limit States:

Beam gross shear capacity (0.4 F_y A_w)= 63.1 Kips ≥ 10.0 Kips **OK**

Beam net shear capacity= 0.3 F_u A_n (AISC ASD Sect. J4)
= 60.8 Kips ≥ 10.0 Kips **OK**

Beam web bearing/tearout capacity (AISC ASD Sect. J3.7):
P_b= 57.6 Kips ≥ 10.0 Kips **OK**

-- Continued --

Beam web connection:

Use three (3) 7/8" ϕ x 2-1/4" w/ 5/32" thick washers
A325-N STD bolts in double shear
User may need to adjust bolt length for other conditions.
Total bolt shear capacity = 75.8 Kips ≥ 10.0 Kips OK

Block shear capacity of coped beam web (AISC ASD Sect. J4):
V_{bs}= 33.1 Kips ≥ 10.0 Kips OK

Bending capacity of coped beam web:
(Manual of Steel Construction Volume II, Connections, Appendix B)
V_{cb}= 48.7 Kips ≥ 10.0 Kips OK (Bending controls)

Girder web connection:

Use six (6) 7/8 in. ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
User may need to adjust bolt length for other conditions.
Total bolt shear capacity = 75.8 Kips ≥ 10.0 Kips OK

Girder web bearing/tearout capacity (AISC ASD Sect. J3.7):
P_b= 129.7 Kips ≥ 10.0 Kips OK
Remaining web bearing/tearout capacity= 119.7 Kips

Framing angles:

2L-SLBB 5x3x1/4 x 10"
A36 Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear capacity= 72.0 Kips ≥ 10.0 Kips OK

Beam side angle leg capacity:
Net shear (AISC ASD Sect. J4):
V_n= 62.5 Kips ≥ 10.0 Kips OK
Block shear (AISC ASD Sect. J4):
V_{bs}= 54.0 Kips ≥ 10.0 Kips OK
Bearing/Tearout (AISC ASD Sect. J3.7):
P_b= 79.0 Kips ≥ 10.0 Kips OK

Girder side angle leg capacity:
Net shear (AISC ASD Sect. J4):
V_n= 62.5 Kips ≥ 10.0 Kips OK
Block shear (AISC ASD Sect. J4):
V_{bs}= 69.4 Kips ≥ 10.0 Kips OK
Bearing/Tearout (AISC ASD Sect. J3.7):
P_b= 91.3 Kips ≥ 10.0 Kips OK

Note:
Number of bolt rows at beam web connection may have been

-- Continued --

increased to satisfy minimum angle length requirements.

-- End --

CONXPRT™ Module I ASD Shear Connection Design Advice
Shear End Plate Knowledge Base
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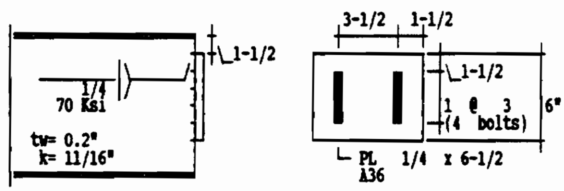
Date: 06/30/94 By: RAK
Job Id: M.E.C.E. PROJECT
Id: B2A TO B2

Beam: W12x14
A36
Girder: W18x60
A36
Shear: 10 Kips

SHEAR END PLATE - BEAM TO GIRDER WEB CONNECTION
Defaults table: SHEARCON.DT1
Beam Elevation: -3"
(with repsect to girder)
Capacity: 15.8 Kips

Girder Web Connection
7/8" ϕ x 2"
A325-N STD Bolts

Girder tw: 0.415"



Connection Description:

Shear end plate, beam to girder web connection.

Beam: W12x14 A36 F_y= 36 ksi F_u= 58 ksi
d= 11.91 in. t_w= 0.2 in.

Girder: W18x60 A36 F_y= 36 ksi F_u= 58 ksi
t_w = 0.415 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 15.8 Kips

Strength Limit States:

Beam gross shear capacity (0.4 F_y A_w)= 34.3 Kips ≥ 10.0 Kips OK

Beam web yield capacity at plate= $(L_p - 2xt_{weld})$ x t_w x 0.4 F_y
= 15.8 Kips ≥ 10.0 Kips OK

Beam web connection:

Use two 1/4 in. - 70 Ksi fillet welds

-- Continued --

Total weld shear capacity = 40.8 Kips ≥ 10.0 Kips OK

Girder web connection:

Use four (4) 7/8 in. ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
User may need to adjust bolt length for other conditions.
Total bolt shear capacity = 50.5 Kips ≥ 10.0 Kips OK

Girder web bearing/tearout capacity (AISC ASD Sect. J3.7):
P_b= 101.1 Kips ≥ 10.0 Kips OK
Remaining web bearing/tearout capacity= 91.1 Kips

Connection Plate:

PL 1/4 x 6-1/2 x 6"
A36 plate F_y= 36 ksi F_u= 58 ksi

Plate gross shear capacity= 43.2 Kips ≥ 10.0 Kips OK
Plate net shear capacity (AISC ASD Sect. J4):
V_n= 35.9 Kips ≥ 10.0 Kips OK
Plate block shear capacity (AISC ASD Sect. J4):
V_{bs}= 41.9 Kips ≥ 10.0 Kips OK
Plate bearing/tearout capacity (AISC ASD Sect. J3.7):
P_b= 60.9 Kips ≥ 10.0 Kips OK

Note:
Number of bolt rows has been increased to satisfy minimum plate length requirements.

-- End --

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Single Plate Knowledge Base
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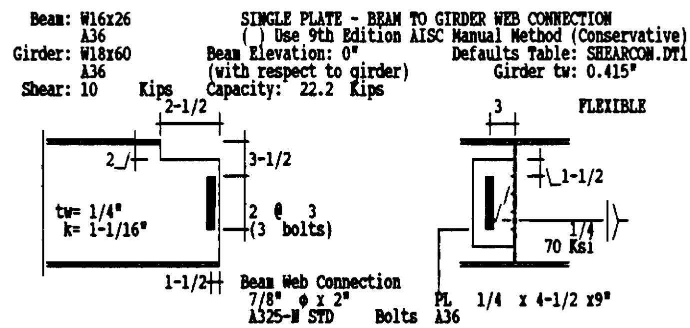
Date: 06/30/94 By: RAK
Job Id: M.E.C.E. PROJECT
Id: B2B TO B2



__Connection Description__:

Single plate (shear tab), beam to girder web connection.
Note:
The girder web is treated as as rotationally flexible element.

Beam: W16x26 A36 F_y= 36 ksi F_u= 58 ksi
d= 15.69 in. t_w= 0.250 in.

Girder: W18x60 A36 F_y= 36 ksi F_u= 58 ksi
t_w = 0.415 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 22.2 Kips

__Strength Limit States__:

Beam gross shear capacity (0.4 F_y A_w)= 49.3 Kips ≥ 10.0 Kips OK

Beam net shear capacity= 0.3 F_u A_n (AISC ASD Sect. J4)
= 47.3 Kips ≥ 10.0 Kips OK

Beam web bearing/tearout capacity (AISC ASD Sect. J3.7):

-- Continued --

(Effective number of bolts= C= 1.76)
P_b= 26.8 Kips ≥ 10.0 Kips OK

Beam web connection:

Use three (3) 7/8 in. ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
User may need to adjust bolt length for other conditions.
r_v= 12.6 Kips
C= 1.76
the value of C was obtained from Table XI in Part 4 of
the AISC ASD Manual using:
b= 3 in. l= 3 in. n= 3
Total bolt shear capacity = C x r_v
= 22.2 Kips ≥ 10.0 Kips OK

Block shear capacity of coped beam web (AISC ASD Sect. J4):
V_{bs}= 28.1 Kips ≥ 10.0 Kips OK

Bending capacity of coped beam web:
(Manual of Steel Construction Volume II, Connections, Appendix B)
V_{cb}= 62.6 Kips ≥ 10.0 Kips OK (Bending controls)

Connection Plate:

PL 1/4 x 4-1/2 x 9"
A36 plate F_y= 36 ksi F_u= 58 ksi

Plate gross shear capacity= 32.4 Kips ≥ 10.0 Kips OK
Plate net shear capacity (AISC ASD Sect. J4):
V_n= 26.9 Kips ≥ 10.0 Kips OK
Plate block shear capacity (AISC ASD Sect. J4):
V_{bs}= 29.9 Kips ≥ 10.0 Kips OK
Plate bearing/tearout capacity (AISC ASD Sect. J3.7):
(Effective number of bolts= C= 1.76)
P_b= 26.8 Kips ≥ 10.0 Kips OK

Girder web connection:

Connection welds are designed for the gross shear capacity
of the plate (32.4 Kips).

Use two 1/4" - 70 Ksi fillet welds
e_w= 3.00 in. C= 1.073
The value of C was obtained from Table XIX in Part 4 of
the AISC ASD Manual using: a= 0.33 k= 0.0

Maximum reaction (based on weld capacity):
R= 38.6 Kips ≥ Plate gross shear capacity = 32.4 Kips OK

-- Continued --

Girder web shear capacity at weld= 107.6 Kips ≥ 10.0 Kips **OK**
Remaining girder web shear capacity= 97.6 Kips

Note:

Number of bolt rows has been increased to satisfy minimum plate length requirements.

-- End --

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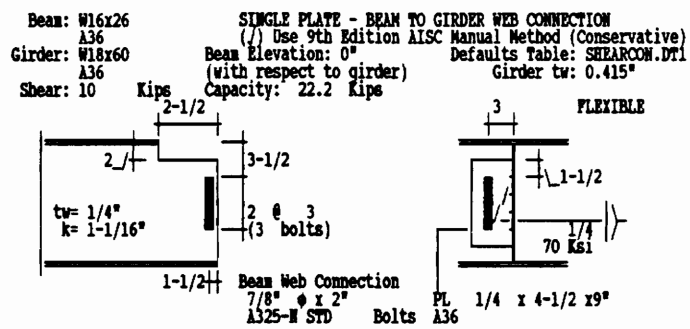
Date: 06/30/94 By: RAK Page 1
Job Id: M.E.C.E. PROJECT
Id: B2B TO B2



Connection Description:

Single plate (shear tab), beam to girder web connection.
Note:
The girder web is treated as as rotationally flexible element.

The method described in the 9th Edition of the AISC-ASD manual was used to design this connection. This method is conservative.

Beam: W16x26 A36 F_y= 36 ksi F_u= 58 ksi
d= 15.69 in. t_w= 0.250 in.

Girder: W18x60 A36 F_y= 36 ksi F_u= 58 ksi
t_w = 0.415 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 22.2 Kips

Strength Limit States:

Beam gross shear capacity (0.4 F_y A_w)= 49.3 Kips ≥ 10.0 Kips OK

-- Continued --

Beam net shear capacity= $0.3\ F_u\ A_n$ (AISC ASD Sect. J4)
= 47.3 Kips ≥ 10.0 Kips OK

Beam web bearing/tearout capacity (AISC ASD Sect. J3.7):
(Effective number of bolts= C= 1.76)
P_b= 26.8 Kips ≥ 10.0 Kips OK

<u>Beam web connection</u>:

Use three (3) 7/8 in. ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
User may need to adjust bolt length for other conditions.
r_v= 12.6 Kips
C= 1.76
the value of C was obtained from Table XI in Part 4 of
the AISC ASD Manual using:
b= 3 in. l= 3 in. n= 3
Total bolt shear capacity = C x r_v
= 22.2 Kips ≥ 10.0 Kips OK

Block shear capacity of coped beam web (AISC ASD Sect. J4):
V_{bs}= 28.1 Kips ≥ 10.0 Kips OK

Bending capacity of coped beam web:
(Manual of Steel Construction Volume II, Connections, Appendix B)
V_{cb}= 62.6 Kips ≥ 10.0 Kips OK (Bending controls)

<u>Connection Plate</u>:

PL 1/4 x 4-1/2 x 9"
A36 plate F_y= 36 ksi F_u= 58 ksi

Plate gross shear capacity= 32.4 Kips ≥ 10.0 Kips OK
Plate net shear capacity (AISC ASD Sect. J4):
V_n= 26.9 Kips ≥ 10.0 Kips OK
Plate block shear capacity (AISC ASD Sect. J4):
V_{bs}= 29.9 Kips ≥ 10.0 Kips OK
Plate bearing/tearout capacity (AISC ASD Sect. J3.7):
(Effective number of bolts= C= 1.76)
P_b= 26.8 Kips ≥ 10.0 Kips OK

<u>Girder web connection</u>:

Connection welds are designed for the gross shear capacity
of the plate (32.4 Kips).

Use two 1/4" - 70 Ksi fillet welds
e_w= 3.00 in. C= 1.073
The value of C was obtained from Table XIX in Part 4 of

-- Continued --

---

the AISC ASD Manual using: a= 0.33 k= 0.0

Maximum reaction (based on weld capacity):
R= 38.6 Kips ≥ Plate gross shear capacity = 32.4 Kips **OK**

Girder web shear capacity at weld= 107.6 Kips ≥ 10.0 Kips **OK**
Remaining girder web shear capacity= 97.6 Kips

Note:

Number of bolt rows has been increased to satisfy minimum plate length requirements.

-- End --

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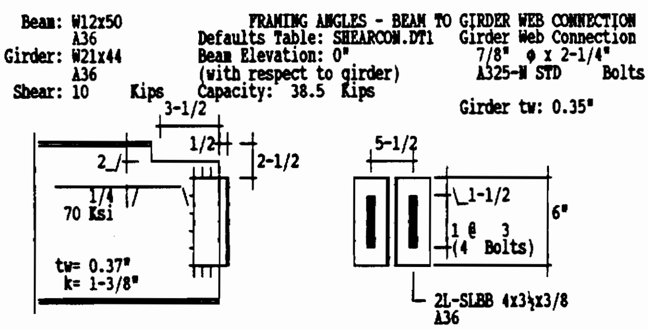
Job Id: M.E.C.E. PROJECT
Id: B3A TO B3



Connection Description:

Framing angles, beam to girder web connection.

Beam: W12x50 A36 F_y = 36 ksi F_u = 58 ksi
d= 12.19 in. t_w = 0.37 in.

Girder: W21x44 A36 F_y = 36 ksi F_u = 58 ksi
t_w = 0.35 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 38.5 Kips

Strength Limit States:

Beam gross shear capacity (0.4 F_y A_w)= 54.3 Kips ≥ 10.0 Kips **OK**

Beam web connection:

Use 1/4 in. - 70 Ksi fillet welds
From AISC-ASD Table XXIII, using k = 0.500 and a = 0.458
Total weld shear capacity = 53.7 Kips ≥ 10.0 Kips **OK**
Beam web capacity at weld = 38.5 Kips ≥ 10.0 Kips **OK**

-- Continued --

Block shear capacity of coped beam web:
(Manual of Steel Construction Volume II, Connections, Page 3-74)
V_{bs}= 46.7 Kips ≥ 10.0 Kips **OK**
Bending capacity of coped beam web:
(Manual of Steel Construction Volume II, Connections, Appendix B)
V_{cb}= 56.4 Kips ≥ 10.0 Kips **OK** (Bending controls)

Girder web connection:

Use four (4) 7/8 in. φ x 2-1/4" w/ 5/32" thick washers
A325-N STD bolts in single shear
User may need to adjust bolt length for other conditions.

Total bolt shear capacity = 50.5 Kips ≥ 10.0 Kips **OK**

Girder web bearing/tearout capacity (AISC ASD Sect. J3.7):
P_b= 85.3 Kips ≥ 10.0 Kips **OK**
Remaining web bearing/tearout capacity= 75.3 Kips

Framing angles:

2L-SLBB 4x3½x3/8 x 6"
A36 Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear capacity= 64.8 Kips ≥ 10.0 Kips **OK**

Girder side angle leg capacity:
Net shear (AISC ASD Sect. J4):
V_n= 53.8 Kips ≥ 10.0 Kips **OK**
Block shear (AISC ASD Sect. J4):
V_{bs}= 61.4 Kips ≥ 10.0 Kips **OK**
Bearing/Tearout (AISC ASD Sect. J3.7):
P_b= 91.3 Kips ≥ 10.0 Kips **OK**

Note:
Number of bolt rows at girder web connection may have been increased to satisfy minimum angle length requirements.

-- End --

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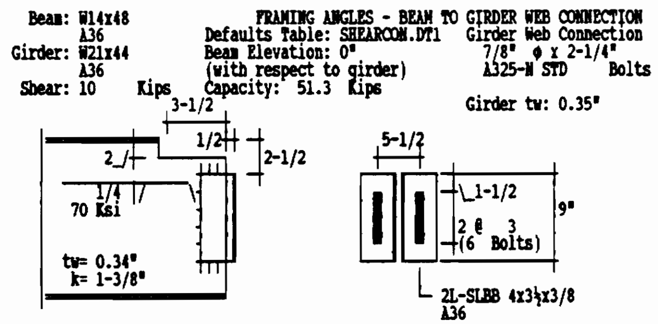

<u>**Connection Description**</u>:

Framing angles, beam to girder web connection.

Beam: W14x48 A36 F_y= 36 ksi F_u= 58 ksi
d= 13.79 in. t_w= 0.34 in.

Girder: W21x44 A36 F_y= 36 ksi F_u= 58 ksi
t_w = 0.35 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 51.3 Kips

<u>**Strength Limit States**</u>:

Beam gross shear capacity (0.4 F_y A_w)= 57.7 Kips ≥ 10.0 Kips OK

<u>Beam web connection</u>:

Use 1/4 in. - 70 Ksi fillet welds
From AISC-ASD Table XXIII, using k = 0.333 and a = 0.322
Total weld shear capacity = 77.8 Kips ≥ 10.0 Kips OK
Beam web capacity at weld = 51.3 Kips ≥ 10.0 Kips OK

-- Continued --

Block shear capacity of coped beam web:
(Manual of Steel Construction Volume II, Connections, Page 3-74)
V_{bs}= 57.6 Kips ≥ 10.0 Kips **OK**
Bending capacity of coped beam web:
(Manual of Steel Construction Volume II, Connections, Appendix B)
V_{cb}= 69.0 Kips ≥ 10.0 Kips **OK** (Bending controls)

Girder web connection:

Use six (6) 7/8 in. φ x 2-1/4" w/ 5/32" thick washers
A325-N STD bolts in single shear
User may need to adjust bolt length for other conditions.

Total bolt shear capacity = 75.8 Kips ≥ 10.0 Kips **OK**

Girder web bearing/tearout capacity (AISC ASD Sect. J3.7):
P_b= 127.9 Kips ≥ 10.0 Kips **OK**
Remaining web bearing/tearout capacity= 117.9 Kips

Framing angles:

2L-SLBB 4x3½x3/8 x 9"
A36 Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear capacity= 97.2 Kips ≥ 10.0 Kips **OK**

Girder side angle leg capacity:
Net shear (AISC ASD Sect. J4):
V_n= 80.7 Kips ≥ 10.0 Kips **OK**
Block shear (AISC ASD Sect. J4):
V_{bs}= 88.0 Kips ≥ 10.0 Kips **OK**
Bearing/Tearout (AISC ASD Sect. J3.7):
P_b= 137.0 Kips ≥ 10.0 Kips **OK**

-- End --

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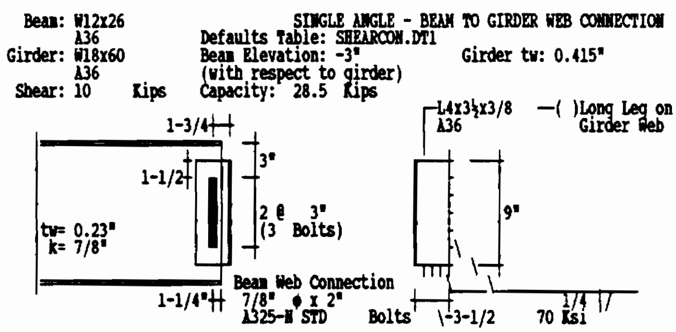

<u>Connection Description</u>:

Single framing angle, beam to girder web connection.

Beam: W12x26 A36 F_y = 36 ksi F_u = 58 ksi
d= 12.22 in. t_w = 0.23 in.

Girder: W18x60 A36 F_y= 36 ksi F_u= 58 ksi
t_w = 0.415 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 28.5 Kips

<u>Strength Limit States</u>:

Beam gross shear capacity (0.4 F_y A_w)= 40.5 Kips ≥ 10.0 Kips **OK**

Beam web bearing/tearout capacity (AISC ASD Sect. J3.7):
P_b= 42.0 Kips ≥ 10.0 Kips **OK**

<u>Beam web connection</u>:

-- Continued --

Use three (3) 7/8" ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
User may need to adjust bolt length for other conditions.
Total bolt shear capacity = 37.9 Kips ≥ 10.0 Kips **OK**

<u>Girder web connection</u>:

Use 1/4 in. - 70 Ksi fillet welds
From Table XXV, ASD-AISC Manual of Steel Construction, 9th Edition, page 4-81, and using k= 0.3889 and a= 0.3472:
C= 0.79
Total weld shear capacity = 28.5 Kips ≥ 10.0 Kips **OK**

Girder web shear capacity at weld= 45.9 Kips ≥ 10.0 Kips **OK**
Remaining girder web shear capacity= 35.9 Kips

<u>Framing angles</u>:

L4x3½x3/8 x 9"
A36 Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear capacity= 48.6 Kips ≥ 10.0 Kips **OK**

Beam side angle leg capacity:
Net shear (AISC ASD Sect. J4):
V_n= 40.4 Kips ≥ 10.0 Kips **OK**
Block shear (AISC ASD Sect. J4):
V_{bs}= 53.0 Kips ≥ 10.0 Kips **OK**
Bearing/Tearout (AISC ASD Sect. J3.7):
P_b= 68.5 Kips ≥ 10.0 Kips **OK**

Warning:
To provide the necessary flexibilty, there should be no weld at the top or heel of the outstanding leg of the connection angle, with the exception of the weld return at the top.

<u>Reference</u>: AISC ASD Manual of Steel Construction, 9^{th} Edition, 1989, page 4-84

-- End --

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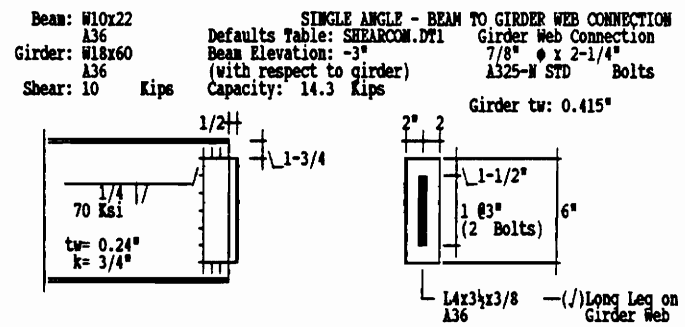

Connection Description:

Framing angles, beam to girder web connection.

Beam: W10x22 A36 F_y= 36 ksi F_u= 58 ksi
d= 10.17 in. t_w= 0.24 in.

Girder: W18x60 A36 F_y= 36 ksi F_u= 58 ksi
t_w = 0.415 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 14.3 Kips

Strength Limit States:

Beam gross shear capacity (0.4 F_y A_w)= 35.1 Kips ≥ 10.0 Kips OK

Beam web connection:

Use 1/4 in. - 70 Ksi fillet welds
From Table XXIII, ASD-AISC Manual of Steel Construction, 9th Edition, page 4-79, and using k= 0.5000 and a= 0.4583:

-- Continued --

C= 1.12
Total weld shear capacity = 26.8 Kips ≥ 10.0 Kips OK
Beam web capacity at weld = 24.9 Kips ≥ 10.0 Kips OK

<u>Girder web connection</u>:

Use two (2) 7/8 in. φ x 2-1/4" w/ 5/32" thick washers
A325-N STD bolts in single shear
User may need to adjust bolt length for other conditions.

r_v= 12.6 Kips
Effective number of bolts: C= 1.14
the value of C was calculated using the ultimate strength
method for determining eccentric loads on fastener groups,
described in page 4-57 of the AISC-ASD Manual, using:
b= 3 in. l= 2.12 in. n= 2
Total bolt shear capacity = 14.3 Kips ≥ 10.0 Kips OK

Girder web bearing/tearout capacity (AISC ASD Sect. J3.7):
(Effective number of bolts: C= 1.14)
P_b= 28.7 Kips ≥ 10.0 Kips OK
Remaining web bearing/tearout capacity= 18.7 Kips

<u>Framing angles</u>:

L4x3½x3/8 x 6"
A36 Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear capacity= 32.4 Kips ≥ 10.0 Kips OK

Girder side angle leg capacity:
Net shear (AISC ASD Sect. J4):
V_n= 26.9 Kips ≥ 10.0 Kips OK
Block shear (AISC ASD Sect. J4):
V_{bs}= 36.8 Kips ≥ 10.0 Kips OK
Bearing/Tearout (AISC ASD Sect. J3.7):
(Effective number of bolts: C= 1.14)
P_b= 25.9 Kips ≥ 10.0 Kips OK
Bending:
On gross section, S = 2.25 in^3
M_a = 0.6 F_y S = 48.6 Kip-in.
Equivalent shear capacity = 22.9 Kips ≥ 10.0 Kips OK
On net section, (y= 3.00 in. I_{net}= 5.00 in^4 S_{net}= 1.67 in^3)
M_a = 0.5 F_u S_{net} = 48.3 Kip-in.
Equivalent shear capacity = 22.8 Kips ≥ 10.0 Kips OK

-- End --

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Beam Flange-Welded, Web-Bolted Knowledge Base
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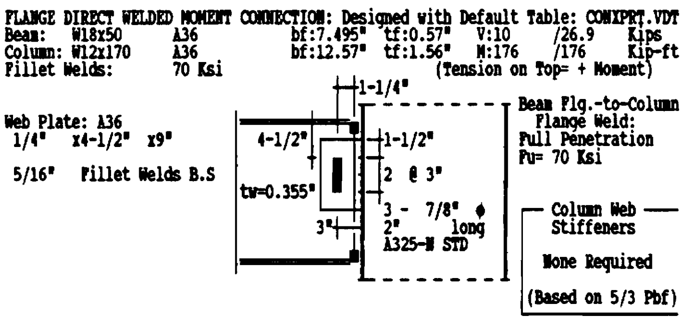
Connection Id: B1 TO C1



A. <u>Connection Description</u>:

Flange direct welded - Web bolted beam to column flange moment connection.

Beam:	W18x50	A36		F_y = 36.00 ksi	F_u = 58 ksi
		d = 17.99 in.		t_w = 0.355 in.	S_x = 88.9 in^3
		b_f = 7.495 in.		t_f = 0.570 in.	T = 15-1/2 in.
Column:	W12x170	A36		F_y = 36.00 ksi	F_u = 58 ksi
		d = 14.03 in.		t_w = 0.960 in.	
		b_f = 12.570 in.		t_f = 1.560 in.	
		k = 2.2500 in.		k_1 = 1.1250 in.	d_c = 9.53 in.

Beam end reaction = 10.0 Kips
Connection shear capacity = 26.9 Kips
Beam end moment = 176.0 Kip-ft.
Beam flange force = $M/(d-t_f)$
= 121.2 Kips
Connection moment capacity = 176.0 Kip-ft.

B. <u>Continuously Braced Beam Flexural Capacity</u>:

$M_a = (\min(b_{fc},b_{fb})/b_{fb}) \times F_b \times S_x$
$= (7.495/7.495) \times 23.76 \times 88.9 / 12$ (Compact Section)

-- Continued --

= 176 Kip-ft ≥ |M|= 176 Kip-ft

If not continuously braced, beam capacity to be verified by designer.

C. Connection Moment Capacity:

Beam Flange-to-Column Flange Connection:
Use full penetration groove welds with F_u= 70 Ksi

D. Connection Shear Capacity:

Beam Gross Shear Capacity:
($0.4\ F_y\ A_w$)= 92.0 Kips ≥ 10.000 Kips OK

Beam Web Bearing/Tearout Capacity:
(AISC ASD Sect. J3.7)
V_b= 64.9 Kips ≥ 10.000 Kips OK

Beam Web Connection:
Use three (3) 7/8 in. ϕ x 2" w/ 5/32" thick washers
A325-N STD bolts in single shear
User may need to adjust bolt length for other conditions.
Total bolt shear capacity = 37.9 Kips ≥ 10.000 Kips OK

Web Connection Plate:
PL 1/4 x 4-1/2 x 9"
A36 plate F_y= 36 ksi F_u= 58 ksi

Plate gross shear capacity= 32.4 Kips ≥ 10.0 Kips OK
Plate net shear capacity (AISC ASD Sect. J4):
V_n= 26.9 Kips ≥ 10.0 Kips OK
Plate block shear capacity (AISC ASD Sect. J4):
V_{bs}= 29.9 Kips ≥ 10.0 Kips OK
Plate bearing/tearout capacity (AISC ASD Sect. J3.7):
V_b= 45.7 Kips ≥ 10.000 Kips OK

Web Plate-to-Column Flange Connection:
Use 5/16" - 70 Ksi fillet welds - both sides

Maximum weld strength:
V= 83.5 Kips ≥ 10.000 Kips OK

Column flange shear capacity at weld= 404.4 Kips ≥ 10.000 Kips OK

E. Column Stiffener Plates:

Design is based on P_{bf}= 5/3 (M/(d-t_f))
= 202.1 Kips

-- Continued --

Column Flange Bending Strength: (AISC-ASD Sect. K1.2)

$P_{fb} = t_f^2 \, F_{yc} / 0.16$
$= 547.6$ Kips $\geq P_{bf} = 202.1$ Kips OK
Column flange bending capacity is adequate.

Column Web Yielding Strength: (AISC-ASD Sect. K1.3)

$P_{wy} = F_{yc} \, t_{wc} \, (t_b + 5\ k)$
$= 408.5$ Kips $\geq P_{bf} = 202.1$ Kips OK
Column web yielding strength is adequate.

Column Web Crippling Capacity: (AISC-ASD Sect. K1.4)

$P_{wc} = 5/3 \times 67.5 \, t_{wc}^2 \, [1 + 3(t_{fb}/d_c)(t_{wc}/t_{fc})^{1.5}] \, \sqrt{(F_{yc} \, t_{fc}/t_{wc})}$
$= 839.7$ Kips $\geq P_{bf} = 202.1$ Kips OK
Column web crippling capacity is adequate.

Column Web Buckling Strength: (AISC-ASD Sect. K1.6)

$P_{wb} = 4100 \, t_{wc}\text{^}3 \, \sqrt{(F_{yc})} / d_c$
$= 2283.8$ Kips $\geq P_{bf} = 202.1$ Kips OK
Column web buckling strength is adequate.

Column stiffener plates are NOT required.

Note:
Number of bolt rows at beam web connection has been increased to accommodate specified web plate length.

Note:
Web plate-to-column flange weld size can be reduced to 1/4 in. if particular care is taken to provide sufficient preheat for soundness of the weld (see AISC-ASD Sect. J2.2b).

Note:
The user should consider using short slots at beam web connection to account for shrinkage caused by beam flange weld.

-- End --

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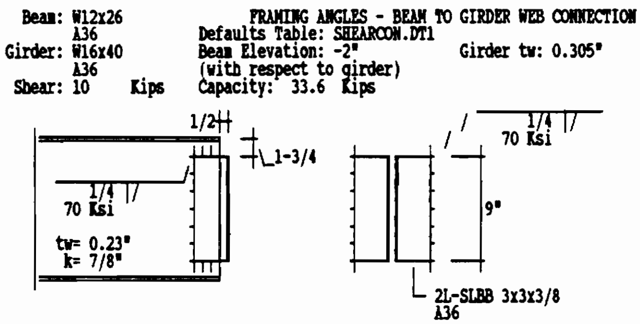

Connection Description:

Framing angles, beam to girder web connection.

Beam: W12x26 A36 F_y= 36 ksi F_u= 58 ksi
d= 12.22 in. t_w= 0.23 in.

Girder: W16x40 A36 F_y= 36 ksi F_u= 58 ksi
t_w = 0.305 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 33.6 Kips

Strength Limit States:

Beam gross shear capacity (0.4 F_y A_w)= 40.5 Kips ≥ 10.0 Kips OK

Beam web connection:

Use 1/4 in. - 70 Ksi fillet welds
From AISC-ASD Table XXIII, using k = 0.278 and a = 0.284
Total weld shear capacity = 75.4 Kips ≥ 10.0 Kips OK
Beam web capacity at weld = 33.6 Kips ≥ 10.0 Kips OK

-- Continued --

Girder web connection:

Use 1/4 in. - 70 Ksi fillet welds
Total weld shear capacity = 42.8 Kips ≥ 10.0 Kips **OK**

Girder web shear capacity at weld= 79.1 Kips ≥ 10.0 Kips **OK**
Remaining girder web shear capacity= 69.1 Kips

Framing angles:

2L-SLBB 3x3x3/8 x 9"
A36 Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear capacity= 97.2 Kips ≥ 10.0 Kips **OK**

Note:
The designer should consider the erectability of this connection.

-- End --

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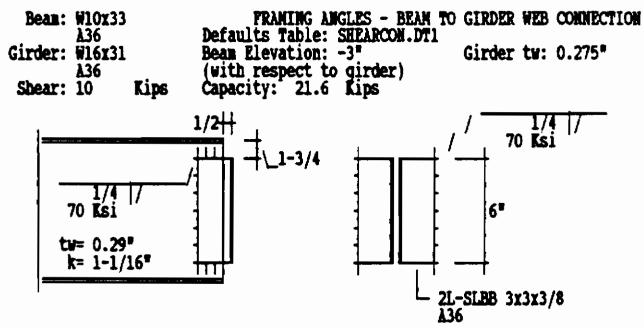

Connection Description:

Framing angles, beam to girder web connection.

Beam: W10x33 A36 F_y = 36 ksi F_u = 58 ksi
d= 9.73 in. t_w = 0.29 in.

Girder: W16x31 A36 F_y = 36 ksi F_u = 58 ksi
t_w = 0.275 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 21.6 Kips

Strength Limit States:

Beam gross shear capacity (0.4 F_y A_w)= 40.6 Kips ≥ 10.0 Kips OK

Beam web connection:

Use 1/4 in. - 70 Ksi fillet welds
From AISC-ASD Table XXIII, using k = 0.417 and a = 0.406
Total weld shear capacity = 52.0 Kips ≥ 10.0 Kips OK
Beam web capacity at weld = 29.2 Kips ≥ 10.0 Kips OK

-- Continued --

Girder web connection:

Use 1/4 in. - 70 Ksi fillet welds
Total weld shear capacity = 21.6 Kips ≥ 10.0 Kips **OK**

Girder web shear capacity at weld= 47.5 Kips ≥ 10.0 Kips **OK**
Remaining girder web shear capacity= 37.5 Kips

Framing angles:

2L-SLBB 3x3x3/8 x 6"
A36 Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear capacity= 64.8 Kips ≥ 10.0 Kips **OK**

Note:
The designer should consider the erectability of this connection.

-- End --

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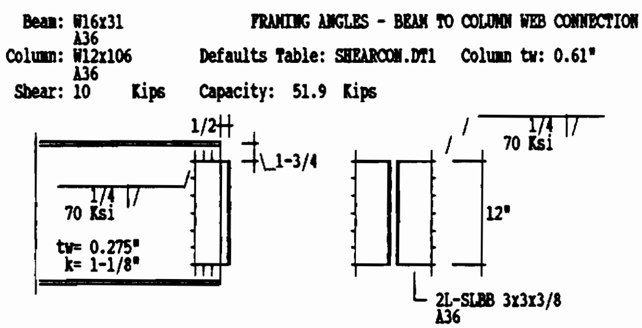


Connection Description:

Framing angles, beam to column web connection.

Beam: W16x31 A36 F_y= 36 ksi F_u= 58 ksi
d= 15.88 in. t_w= 0.275 in.

Column: W12x106 A36 F_y= 36 ksi F_u= 58 ksi
t_w = 0.61 in.

Beam end reaction= 10.0 Kips
Connection Capacity= 51.9 Kips

Strength Limit States:

Beam gross shear capacity (0.4 F_y A_w)= 62.9 Kips ≥ 10.0 Kips OK

Beam web connection:

Use 1/4 in. - 70 Ksi fillet welds
From AISC-ASD Table XXIII, using k = 0.208 and a = 0.220
Total weld shear capacity = 97.3 Kips ≥ 10.0 Kips OK
Beam web capacity at weld = 51.9 Kips ≥ 10.0 Kips OK

-- Continued --

Column web connection:

Use 1/4 in. - 70 Ksi fillet welds
Total weld shear capacity = 66.2 Kips ≥ 10.0 Kips **OK**

Column web shear capacity at weld= 210.8 Kips ≥ 10.0 Kips **OK**
Remaining column web shear capacity= 200.8 Kips

Framing angles:

2L-SLBB 3x3x3/8 x 12"
A36 Angles F_y= 36 ksi F_u= 58 ksi

Angle gross shear capacity= 129.6 Kips ≥ 10.0 Kips **OK**

Note:
The designer should consider the erectability of this connection.

-- End --

APPENDIX C
SCULPTURE MATERIAL LISTS

Table C.1 - Sculpture Material List

SECTION	LENGTH
W21X50	1' - 6"
W21X44	5' - 0"
W18X60	10' - 0"
W18X50	5' - 0"
W18X40	1' - 6"
W16X40	4' - 0"
W16X36	4' - 0"
W16X26	1' - 6"
W16X31	4' - 0"
W16X26	1' - 6"
W14X48	1' - 6"
W14X34	1' - 6"
W12X170	7' - 6"
W12X106	5' - 6"
W12X26	1' - 6"
W10X33	1' - 6"
W10X22	1' - 6"
C9X15	5' - 3"
WT5X15	0' - 9"
4" STD PIPE	3' - 10 3/4"

SECTION	LENGTH
L6X6X3/8	0' - 5 1/2"
L5X3 1/2X3/8	9' - 2 1/2"
L5X3X1/4	0' - 10"
L4X4X1/2	1' - 0"
L4X4X3/8	10' - 8"
L4X3 1/2X3/8	2' - 6"
L3X3X3/8	4' - 6"
L3X3X1/4	16' - 4"
PL1/4X4 1/2	1' -6"
PL1/4X6	1' - 10 1/2"
PL1/4X6 1/2	1' - 3"
PL3/8X3 1/2	1' - 5"
PL1/2X4	1' - 1/2"
PL1/2X6 1/2	3' - 1"
PL1/2X10	5' - 1 1/4"
PL1/2X1'-1"	1' - 3 1/2"
PL5/8X10 1/2	1' - 1"
PL3/4X4	5' - 0"
PL3/4X10 7/8	4' - 6"
PL1X7 1/2	2' - 3 1/4"
PL1X8 1/2	1' - 9"
1 3/8" dia Th Rod	4' -4"

Table C.2 - Beam Member Material List

MARK	SECTION	LENGTH	QUANTITY
B1	W18X50	5' - 0"	1
B1A	W14X34	1' - 6"	1
B1B	W18X40	1' - 6"	1
B2	W18X60	5' - 0"	1
B2A	W12X14	1' - 6"	1
B2B	W16X26	1' - 6"	1
B3	W21X44	2' - 6"	2
B3A	W21X50	1' - 6"	1
B3B	W14X48	1' - 6"	1
B4	W18X60	5' - 0"	1
B4A	W12X26	1' - 6"	1
B4B	W10X22	1' - 6"	1
B5	W16X36	4' - 0"	1
B5A	12K1	3' - 0"	1
B5B	12K1	3' - 0"	1
B6	W16X40	4' - 0"	1
B6A	W12X26	1' - 6"	1
B7-TRUSS			
U1	L5 X 3 1/2 X3/8	4' - 7 1/4"	2
L1	L4 X 4 X 3/8	4' - 7"	2
D1	L3 X 3 X 1/4	3' - 9 5/8"	2
D2	L3 X 3 X 1/4	1' - 0"	2
U1L1	L3 X 3 X 1/4	3' - 4 3/8"	2
B8	W16X31	4' - 0"	1
B8A	W10X33	1' - 6"	1
B9	C9X15	5' - 3"	1

Table C.3 - Column Member Material List

MARK	SECTION	LENGTH	QUANTITY
C1	W12X170	7' - 6"	1
C2	W12X106	5' - 6"	1
C3	4" STD PIPE	3' - 10 3/4"	1

Table C.4 - Connection Material List

MARK	SECTION	LENGTH	QUANTITY	DESCRIPTION
ta	WT5X15	0' - 9"	1	B1A TO B1
aa	L5 X 3 X 1/4	0' - 10"	1	BIB TO B1
pa	PL1/4 X 6 1/2	0' - 6"	1	B2A TO B2
pb	PL1/4 X 4 1/2	0' - 9"	1	B2B TO B2
ab	L4 X 3 1/2 X 3/8	0' - 6"	1	B3A TO B3
ac	L4 X 3 1/2 X 3/8	0' - 9"	1	B3B TO B3
pc	PL1/4 X 6 1/2	1' - 3"	2	B3 SPLICE
pd	PL1/2 X 6 1/2	1' - 6 1/2"	2	B3 SPLICE
ad	L4 X 3 1/2 X 3/8	0' - 9"	1	B4A TO B4
ae	L4 X 3 1/2 X 3/8	0' - 6"	1	B4B TO B4
pe	PL3/8 X 3 1/2	0' - 8 1/2"	2	B4 STIFFENERS
pf	PL1/4 X 4 1/2	0' - 9"	1	B1 TO C1
pg	PL1/2 X 10	1' - 6 1/2"	1	B2 TO C1
ph	PL3/4 X 10 7/8	1' - 3 1/2"	2	B2 TO C1
pi	PL1 X 7 1/2	2' - 3 1/4"	1	B3 TO C1
pj	PL1/2 X 10	1' - 4 3/4"	1	B4 TO C1
pk	PL3/4 X 10 7/8	0' - 6 1/2"	2	B4 TO C1
af	L3 X 3 X 3/8	0' - 9"	2	B6A TO B6
pl	PL1/2 X 13	1' - 3 1/2"	1	U1 TO C2
pm	PL1/2 X 4	1' - 1/2"	1	L1 TO C2
pn	PL1/2 X 9	0' - 10"	1	U1L1 TO U1
po	PL1/2 X 10	1' - 5"	1	U1L1 TO L1
ag	L3 X 3 X 3/8	0' - 6"	2	B8A TO B8
pp	PL1/4 X 6	0' - 8"	2	B9 TO B5/B8
ah	L4 X 4 X 3/8	0' - 5 1/2"	1	B5 TO C2
ai	L6 X 6 X 3/8	0' - 5 1/2"	1	B5 TO C2

Table C.4 - Connection Material List (continued)

MARK	SECTION	LENGTH	QUANTITY	DESCRIPTION
aj	L4 X 4 X 3/8	0' - 6"	1	B6 TO C2
ak	L4 X 4 X 3/8	0' - 6 1/2"	1	B6 TO C2
al	L3 X 3 X 3/8	1' - 0"	2	B8 TO C2
pq	PL5/8 X 5 1/2	0' - 10 1/2"	1	C3 BASE PLATE
pr	PL5/8 X 7 1/2	0' - 10 1/2"	1	C3 BASE PLATE
ps	PL5/8 X 9	0' - 9"	1	COL SPLICE
pt	PL1 X 8 1/2	1' - 9"	1	COL SPLICE
pu	PL1 1/2 X 18	2' - 2"	1	C1 BASE PLATE
pv	PL3/4 X 4	0' - 8"	6	C1 BASE PLATE
am	L4 X 4 X 1/2	1' - 0"	1	C1BASE PLATE
pw	PL3/4 X 4	1' - 0"	1	B6 HANGER PL
-	1 3/8" Thr Rod	4' - 4"	1	B6 HANG ROD
cl	#3 Clevis	-	1	B6 HANG CONN
-	1 1/2" dia Pin	-	1	B6 HANG CONN
wh	Hillside Washer	-	1	B6 HANG CONN
nt	Nut for 1 3/8" Rod	-	1	B6 HANG CONN

Note: Connection bolts and anchor bolts are not given
Connection bolts are 7/8" diam. A325-N

APPENDIX D
SCULPTURE SHOP DRAWINGS

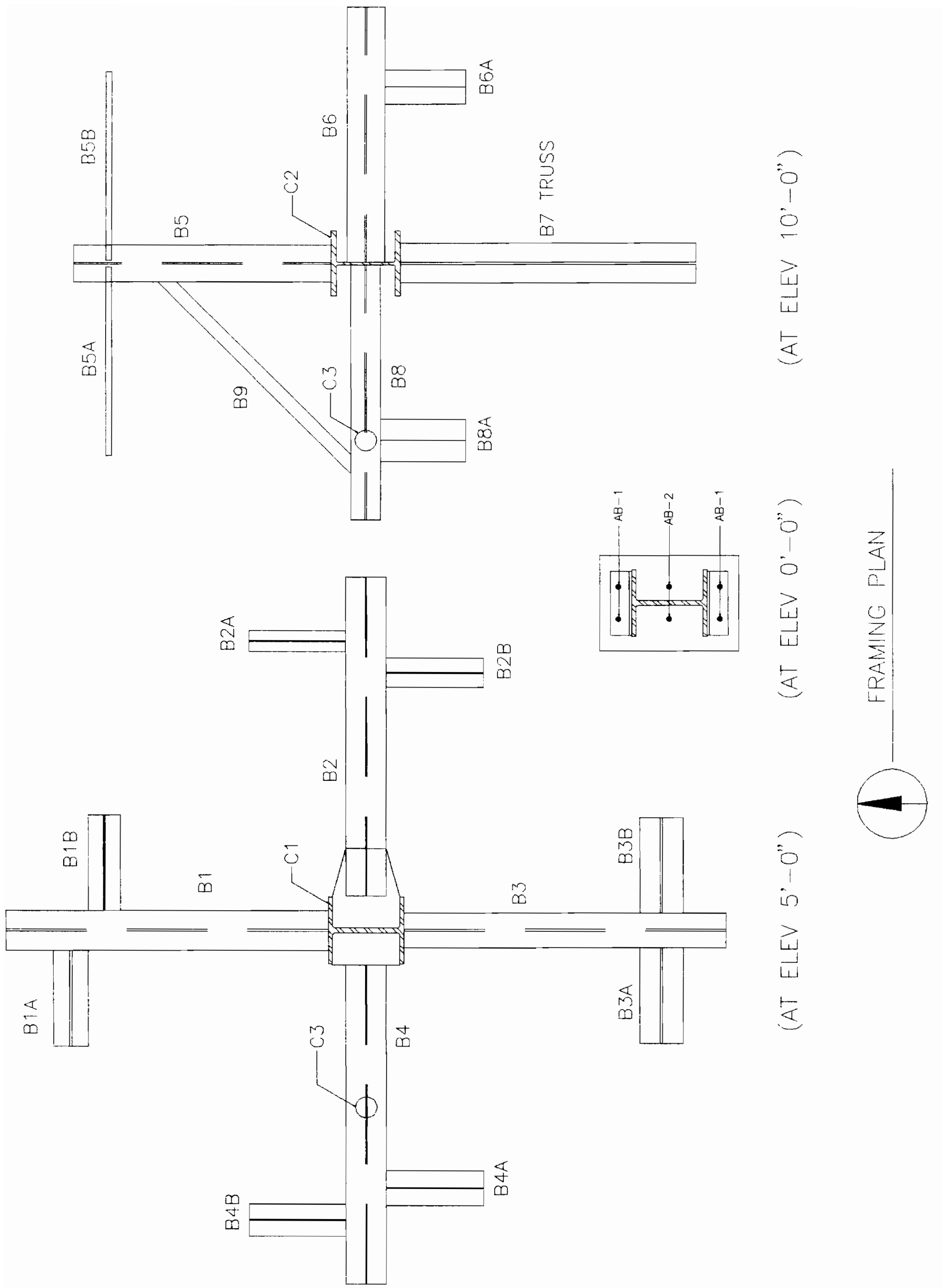

B5B
B5
B5A
B9
C2
C3
B6
B8
B6A
B8A
B7 TRUSS
(AT ELEV 10'-0")
AB-1
AB-2
AB-1
(AT ELEV 0'-0")
FRAMING PLAN
B1B
B1
B1A
C1
B2A
B2
B2B
C3
B4
B4B
B4A
B3
B3B
B3A
(AT ELEV 5'-0")

Figure D.1

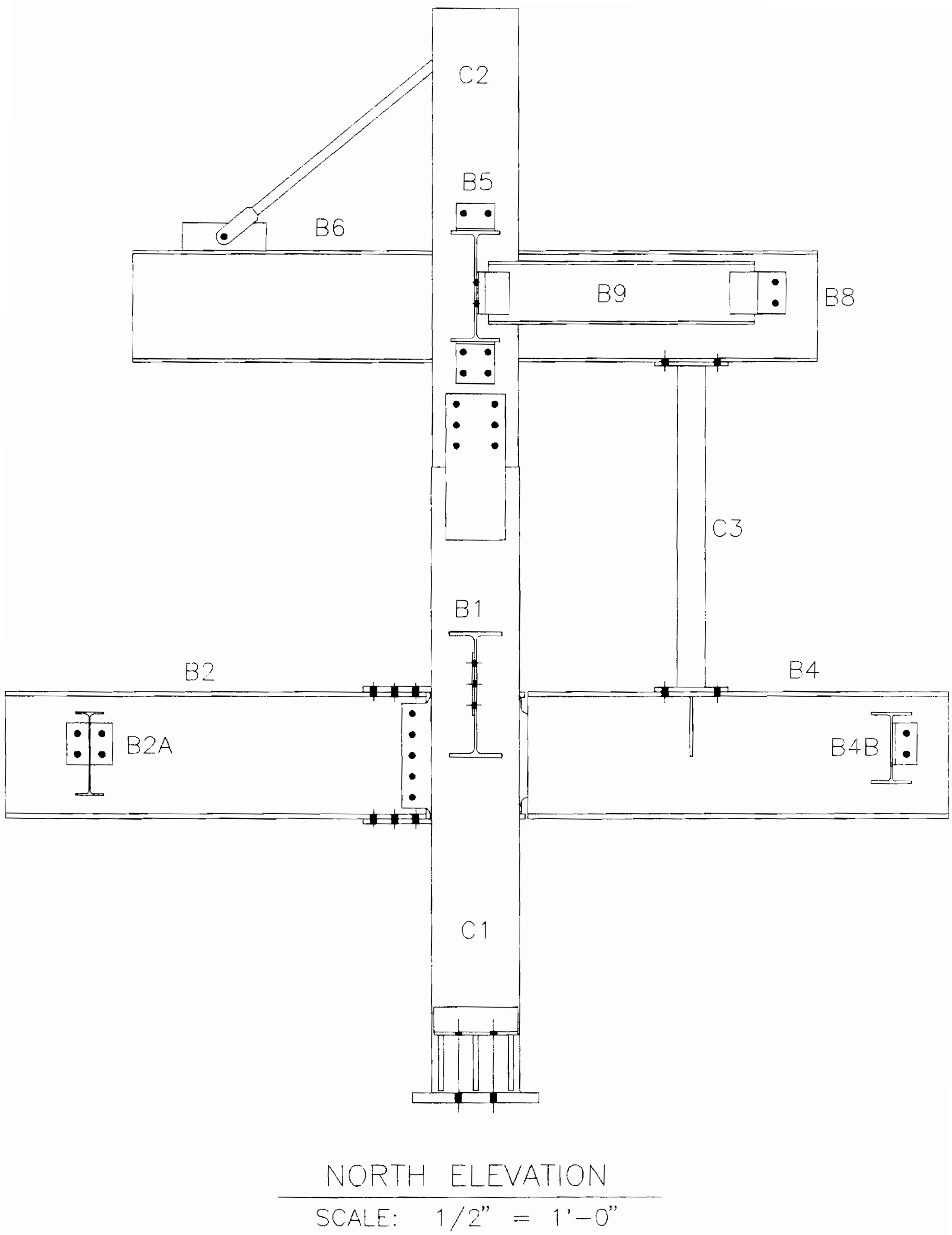


Figure D.2

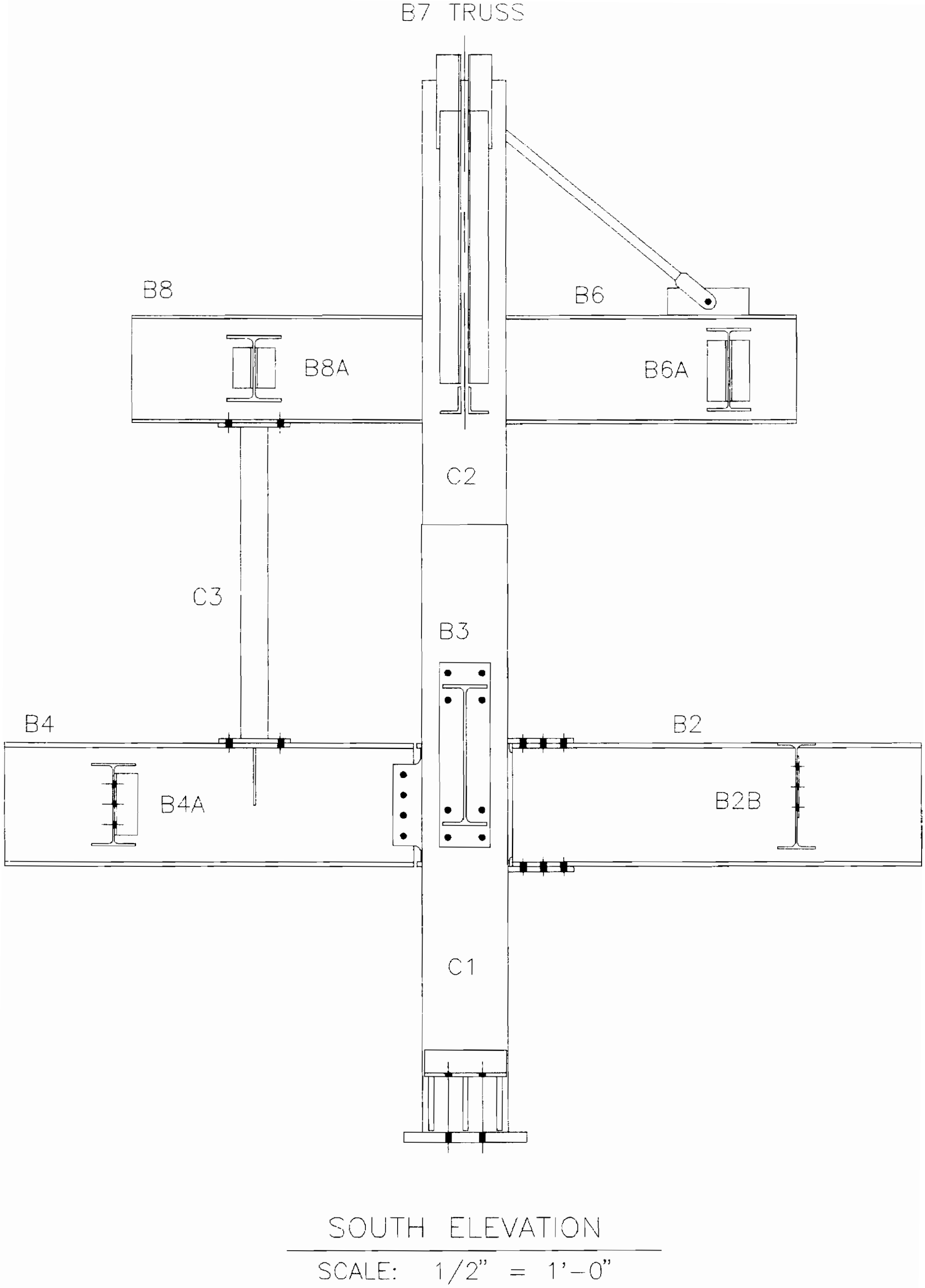
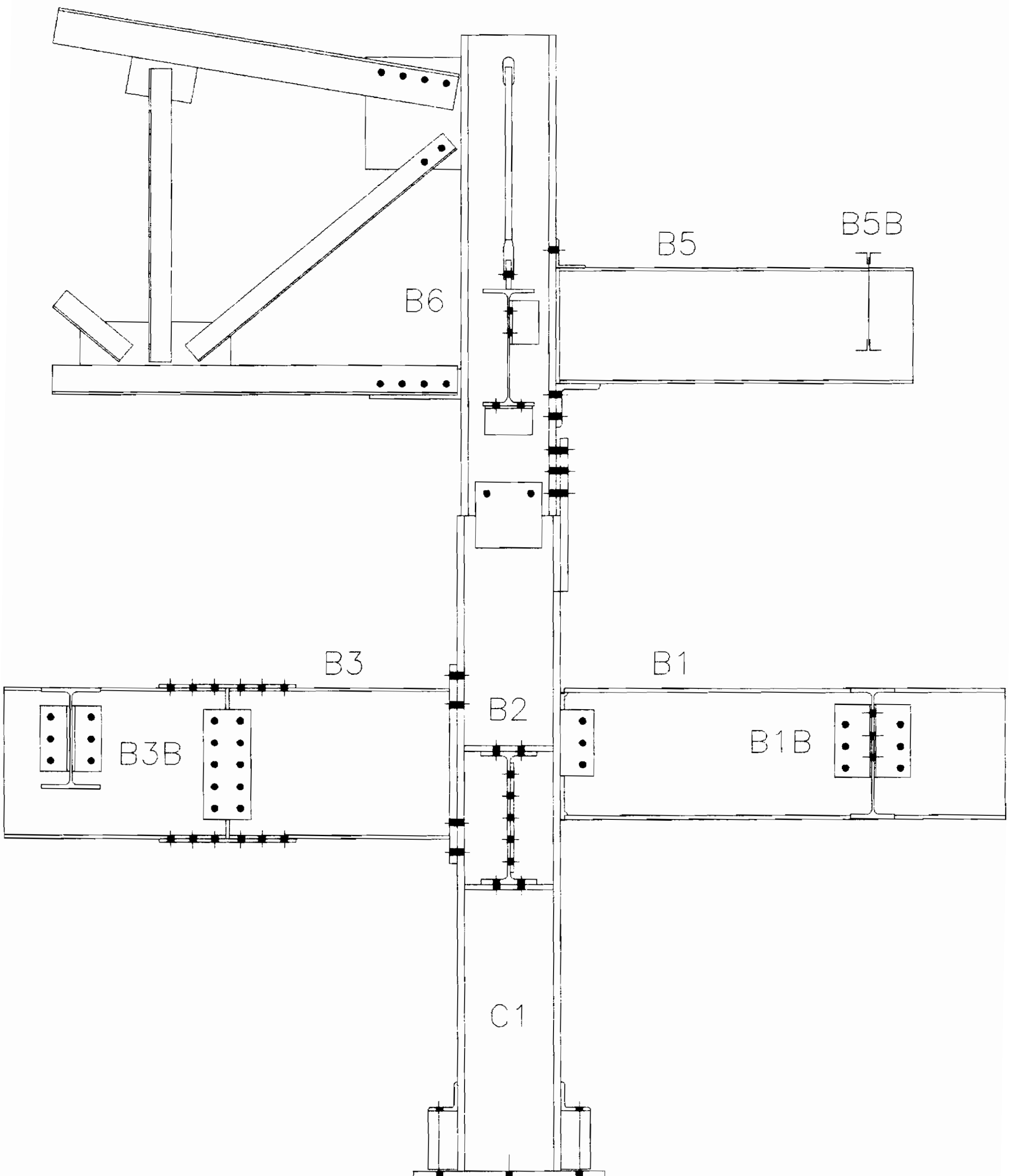

B7 TRUSS
B8
B6
B8A
B6A
C2
C3
B3
B4
B2
B4A
B2B
C1
SOUTH ELEVATION
SCALE: 1/2" = 1'-0"

Figure D.3



EAST ELEVATION

SCALE: 1/2" = 1'-0"

Figure D.4

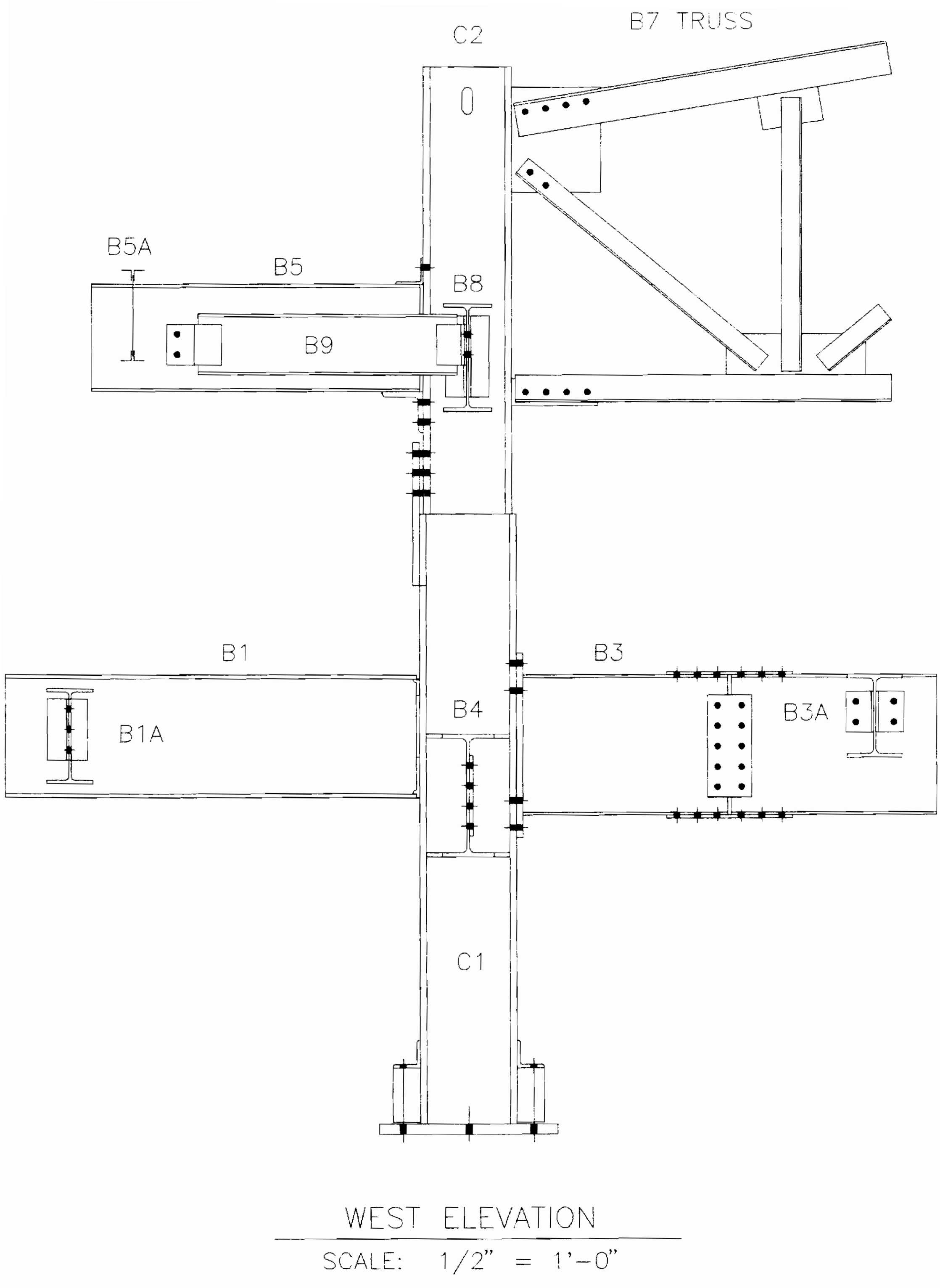
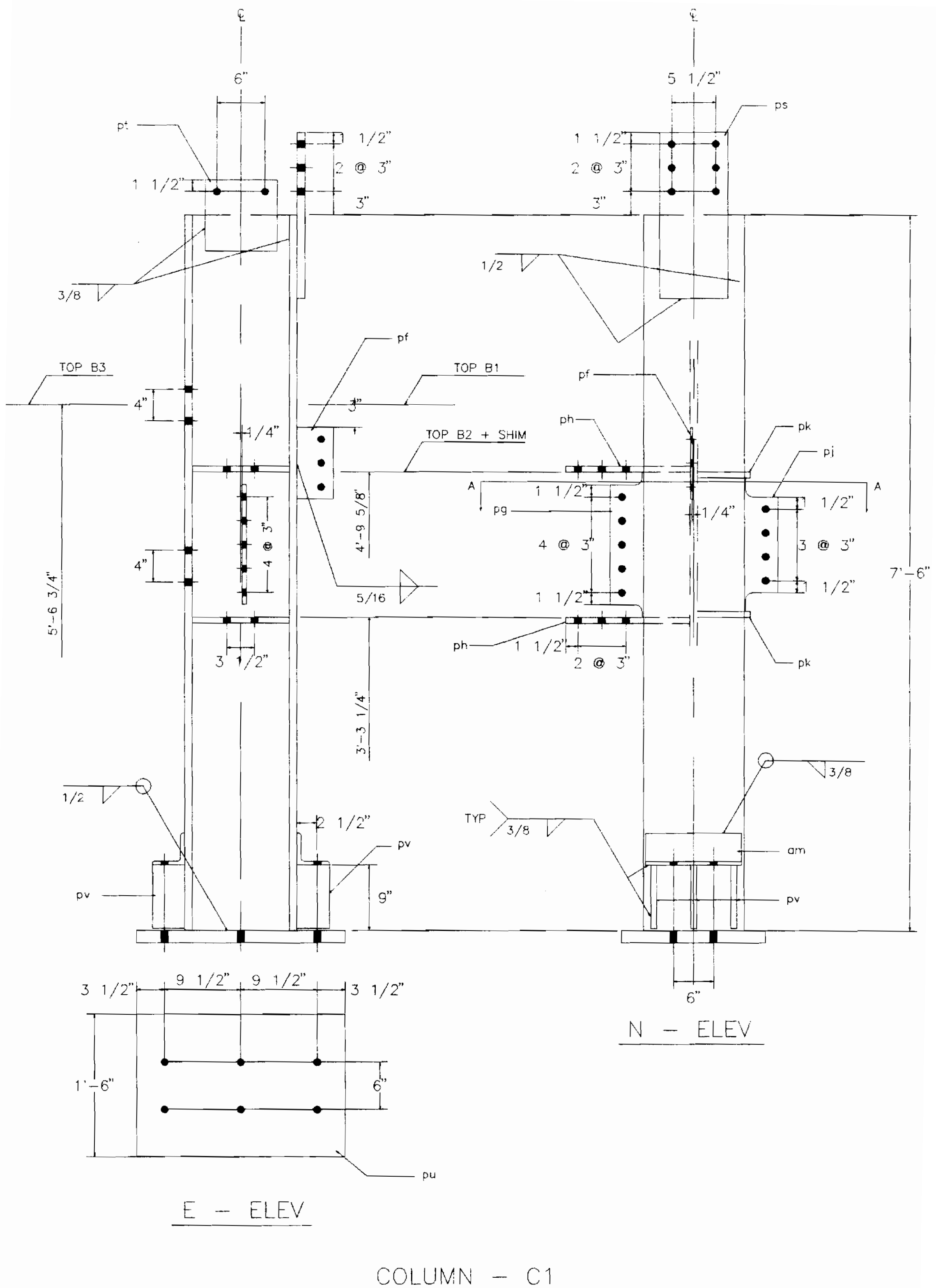

B7 TRUSS
C2
B5A
B5
B8
B9
B1
B3
B4
B1A
B3A
C1
WEST ELEVATION
SCALE: 1/2" = 1'-0"

Figure D.5



COLUMN - C1
E - ELEV
N - ELEV
TOP B3
TOP B1
TOP B2 + SHIM
pt
ps
pf
ph
pk
pi
pg
pv
pu
am
TYP
7'-6"
5'-6 3/4"
4'-9 5/8"
3'-3 1/4"
1'-6"

Figure D.6

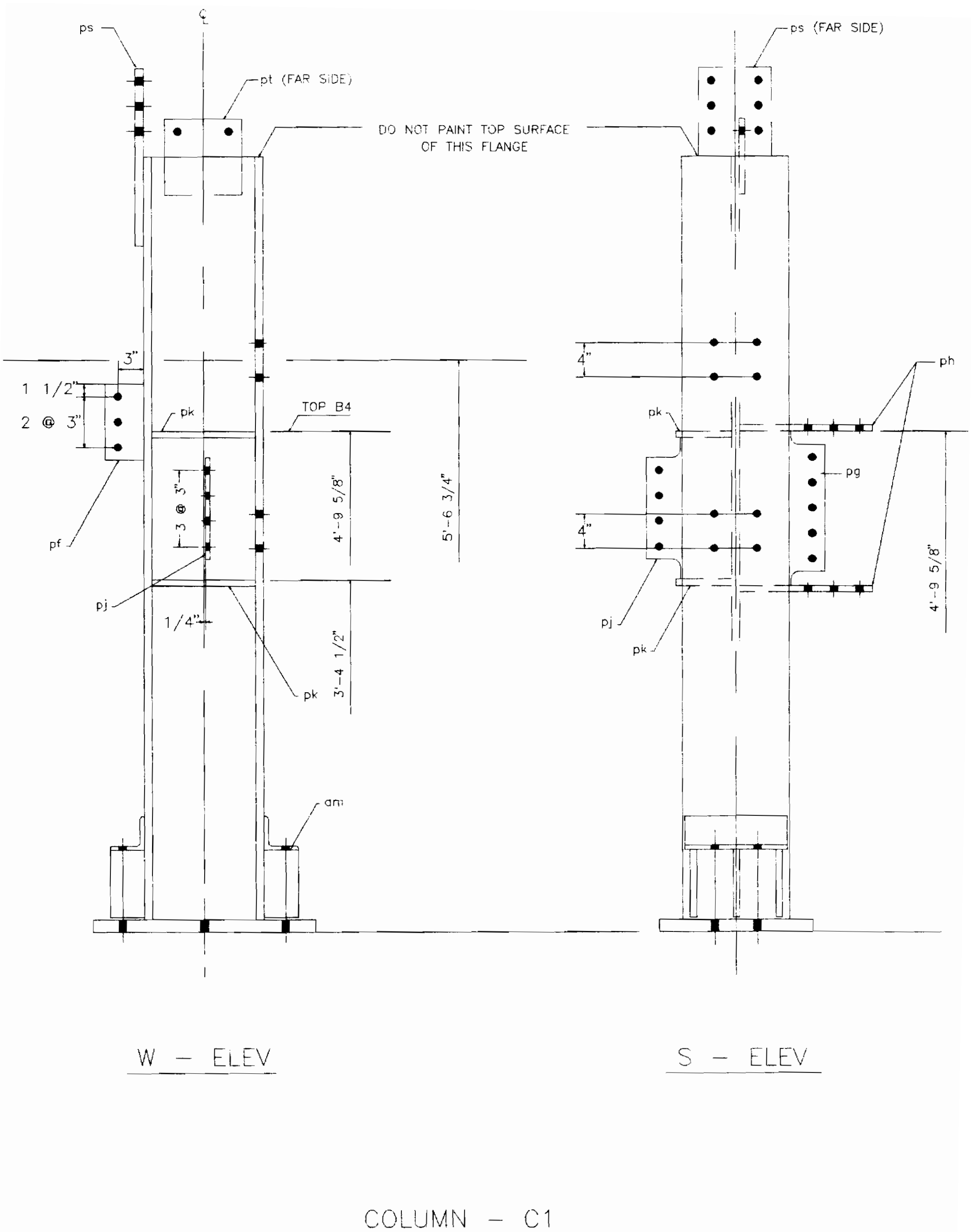
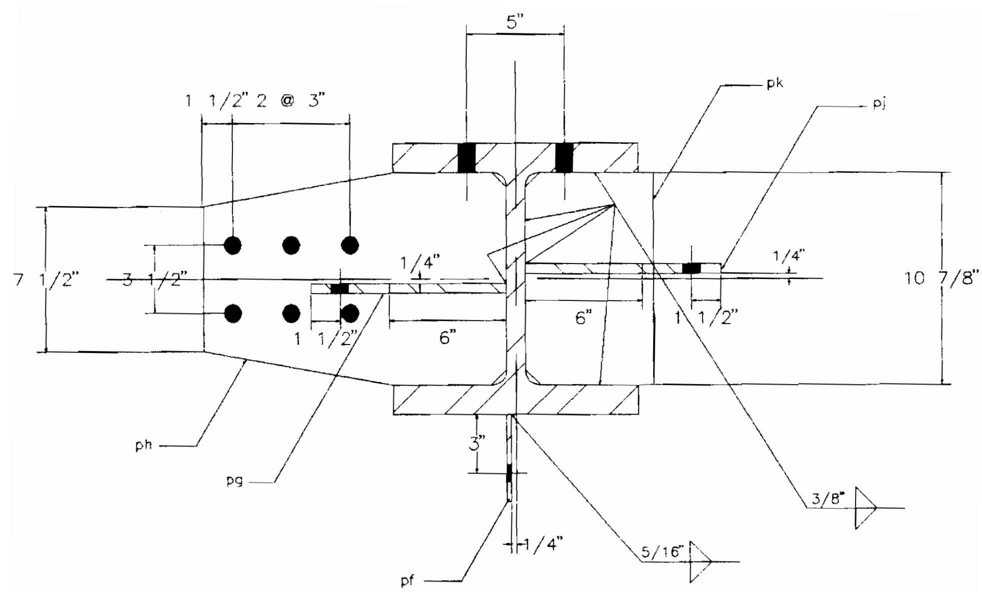

ps
pt (FAR SIDE)
ps (FAR SIDE)
DO NOT PAINT TOP SURFACE
OF THIS FLANGE
3"
1 1/2"
2 @ 3"
pk
TOP B4
3 @ 3"
4'-9 5/8"
5'-6 3/4"
pf
pj
1/4"
pk
3'-4 1/2"
dm
4"
ph
pk
pg
4"
4'-9 5/8"
pj
pk
W - ELEV
S - ELEV
COLUMN - C1

Figure D.7



SECTION A – A

COLUMN – C1

Figure D.8

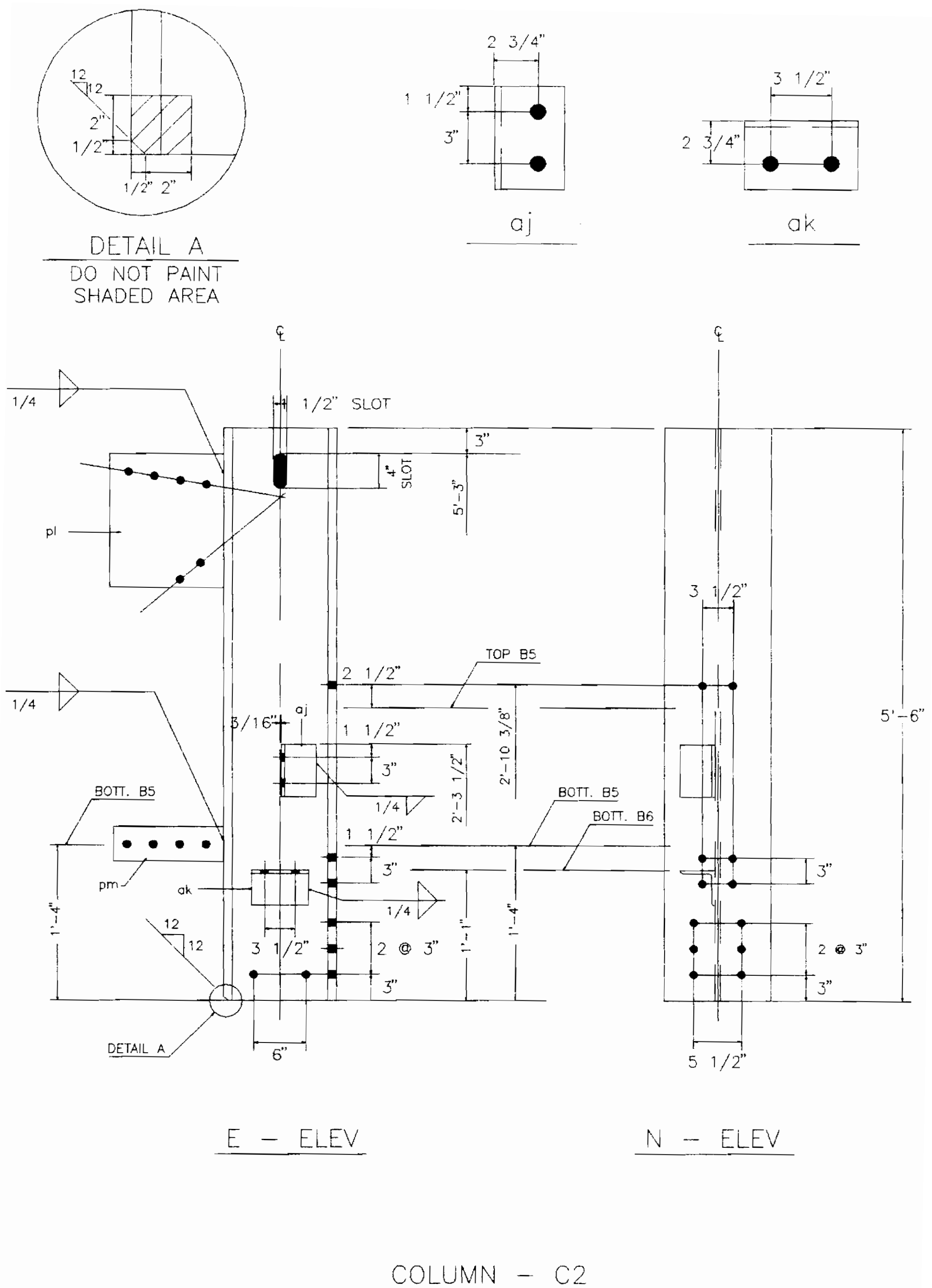
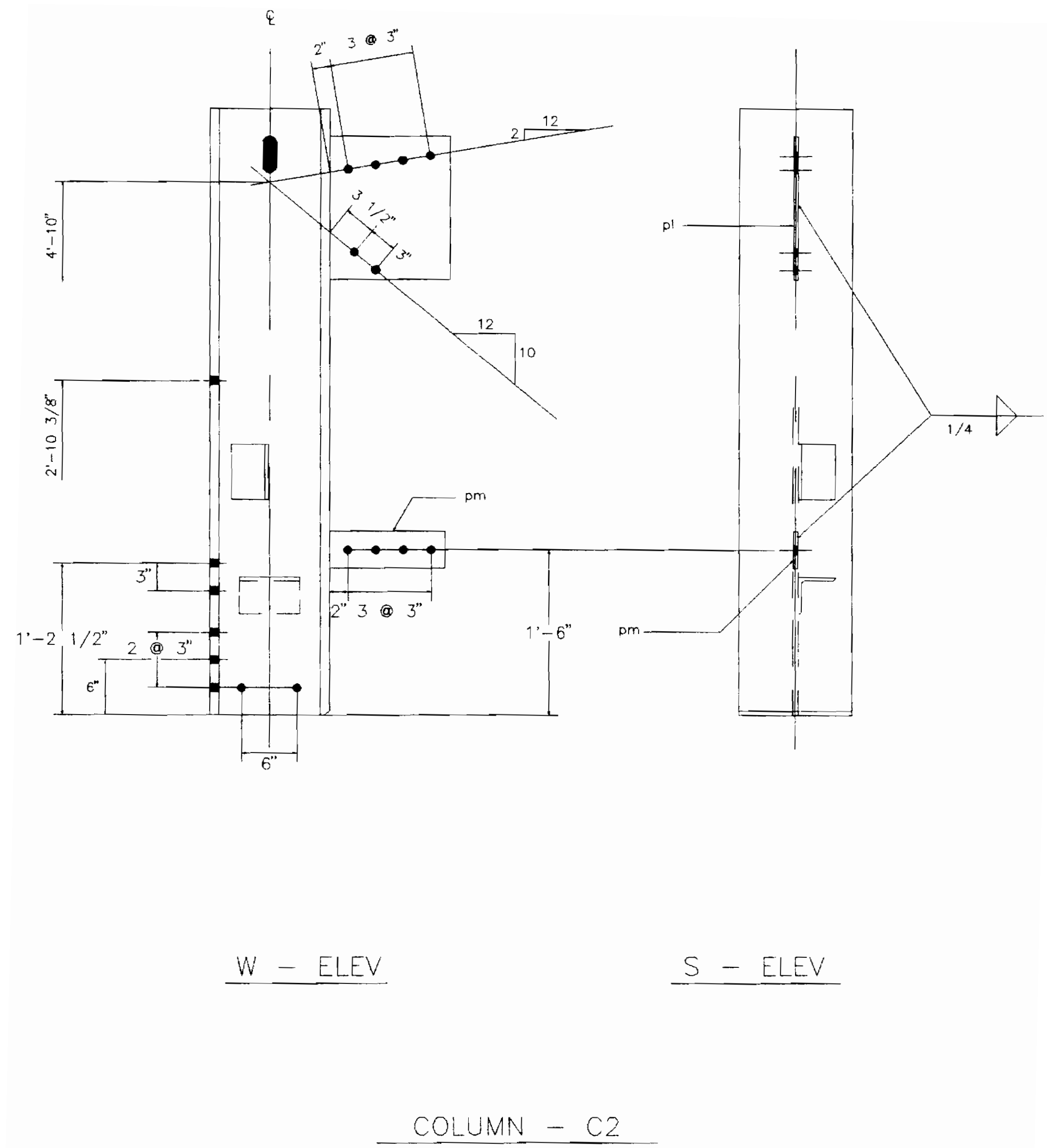
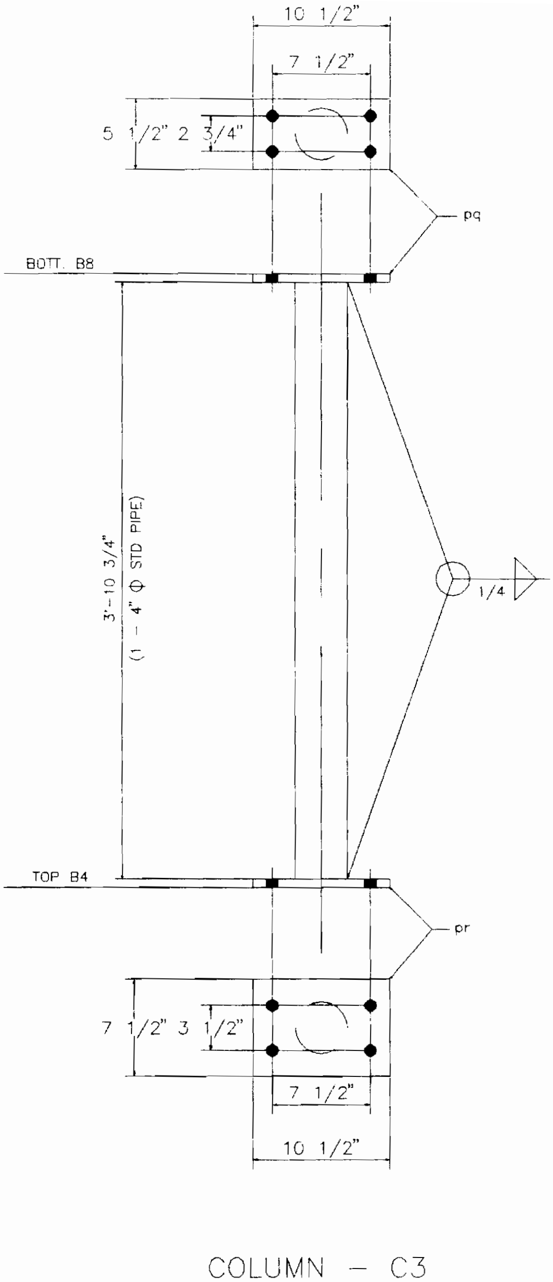

DETAIL A
DO NOT PAINT SHADED AREA
aj
ak
1/2" SLOT
4" SLOT
5'-3"
pl
TOP B5
BOTT. B5
BOTT. B6
pm
DETAIL A
2 @ 3"
5'-6"
E – ELEV
N – ELEV
COLUMN – C2

Figure D.9

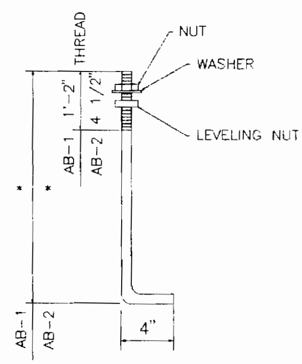


℄
2"
3 @ 3"
2
12
4'-10"
3 1/2"
3"
12
10
pl
1/4
2'-10 3/8"
pm
3"
2" 3 @ 3"
1'-2 1/2"
2 @ 3"
1'-6"
pm
6"
6"
W - ELEV
S - ELEV
COLUMN - C2

Figure D.10



COLUMN - C3



ANCHOR BOLT - AB1

ANCHOR BOLT - AB2

NOT TO SCALE

* LENGTH AS REQUIRED TO SUSTAIN LATERAL FORCES (WIND, SEISMIC)

Figure D.11

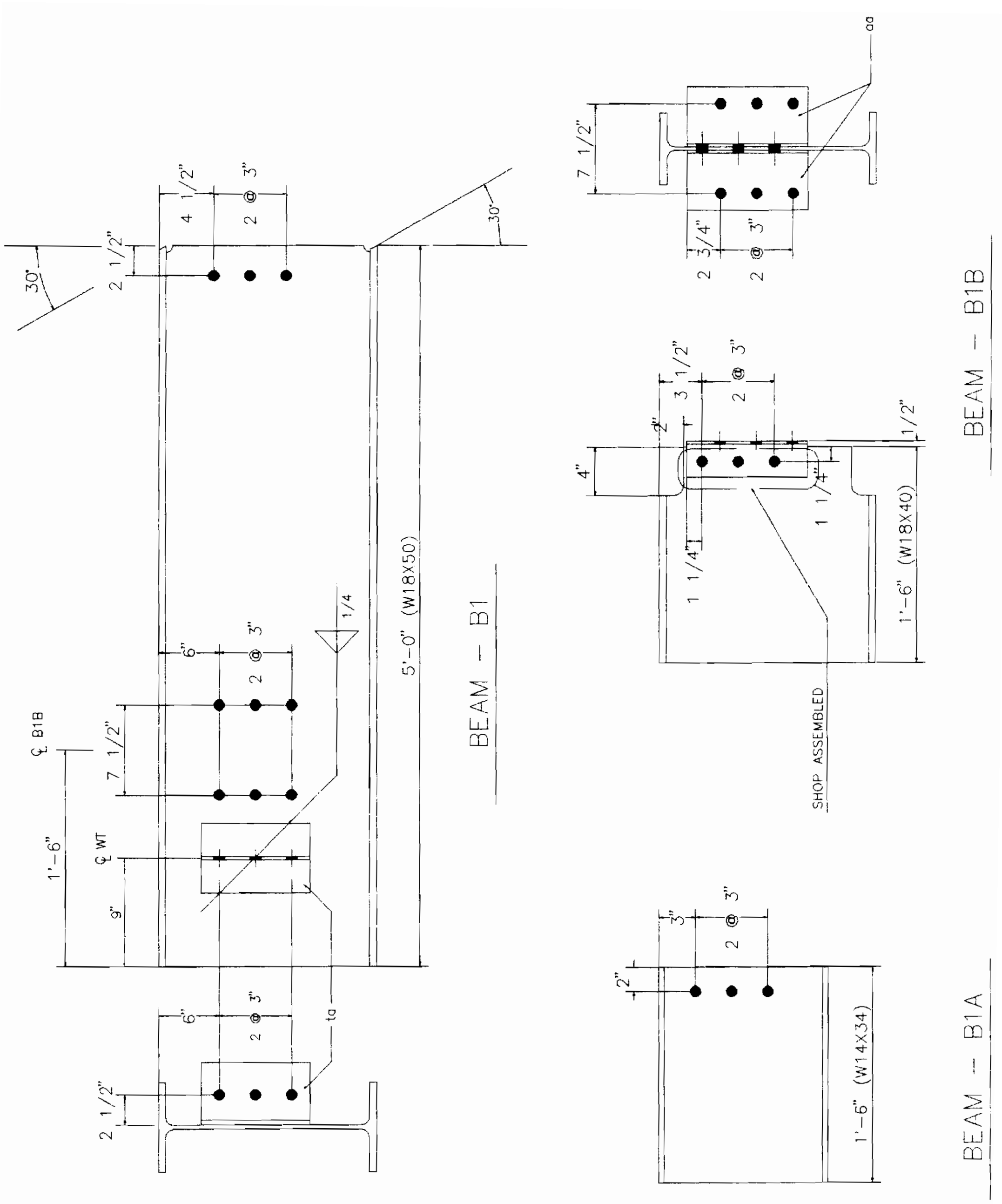

BEAM - B1
5'-0" (W18X50)
BEAM - B1B
1'-6" (W18X40)
SHOP ASSEMBLED
BEAM - B1A
1'-6" (W14X34)

Figure D.12

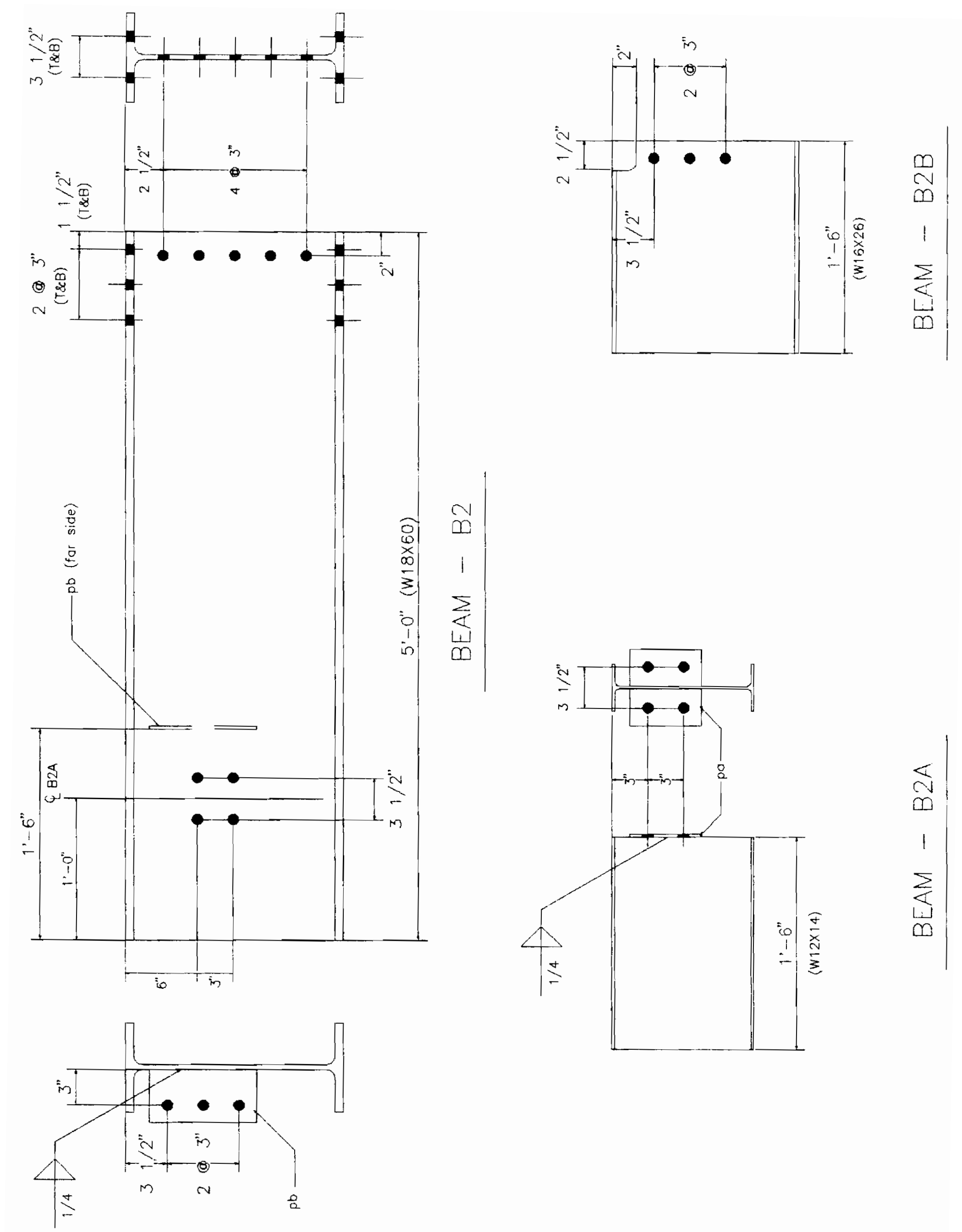

BEAM - B2
5'-0" (W18X60)
pb (far side)
BEAM - B2A
1'-6"
(W12X14)
pa
BEAM - B2B
1'-6"
(W16X26)

Figure D.13

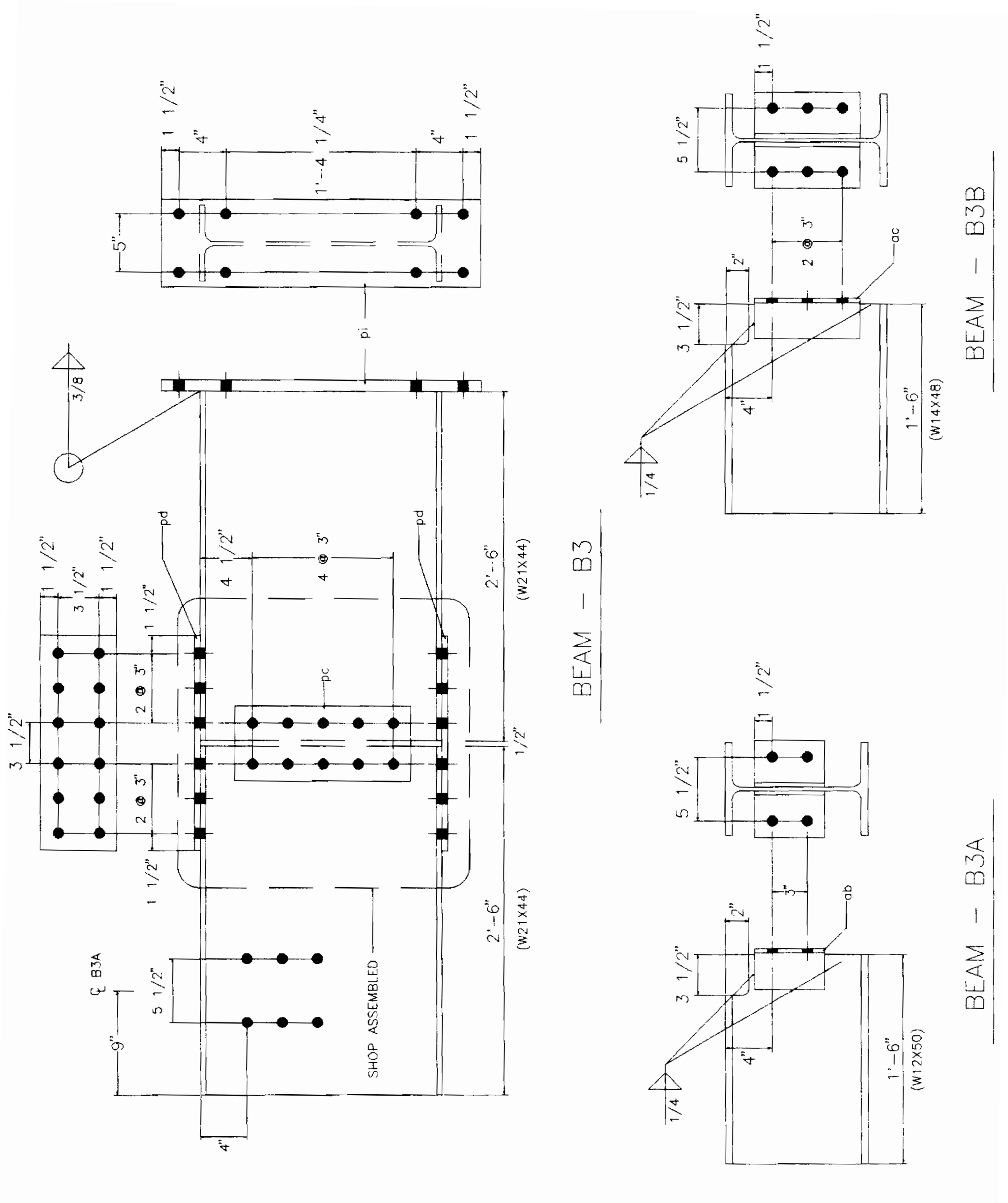
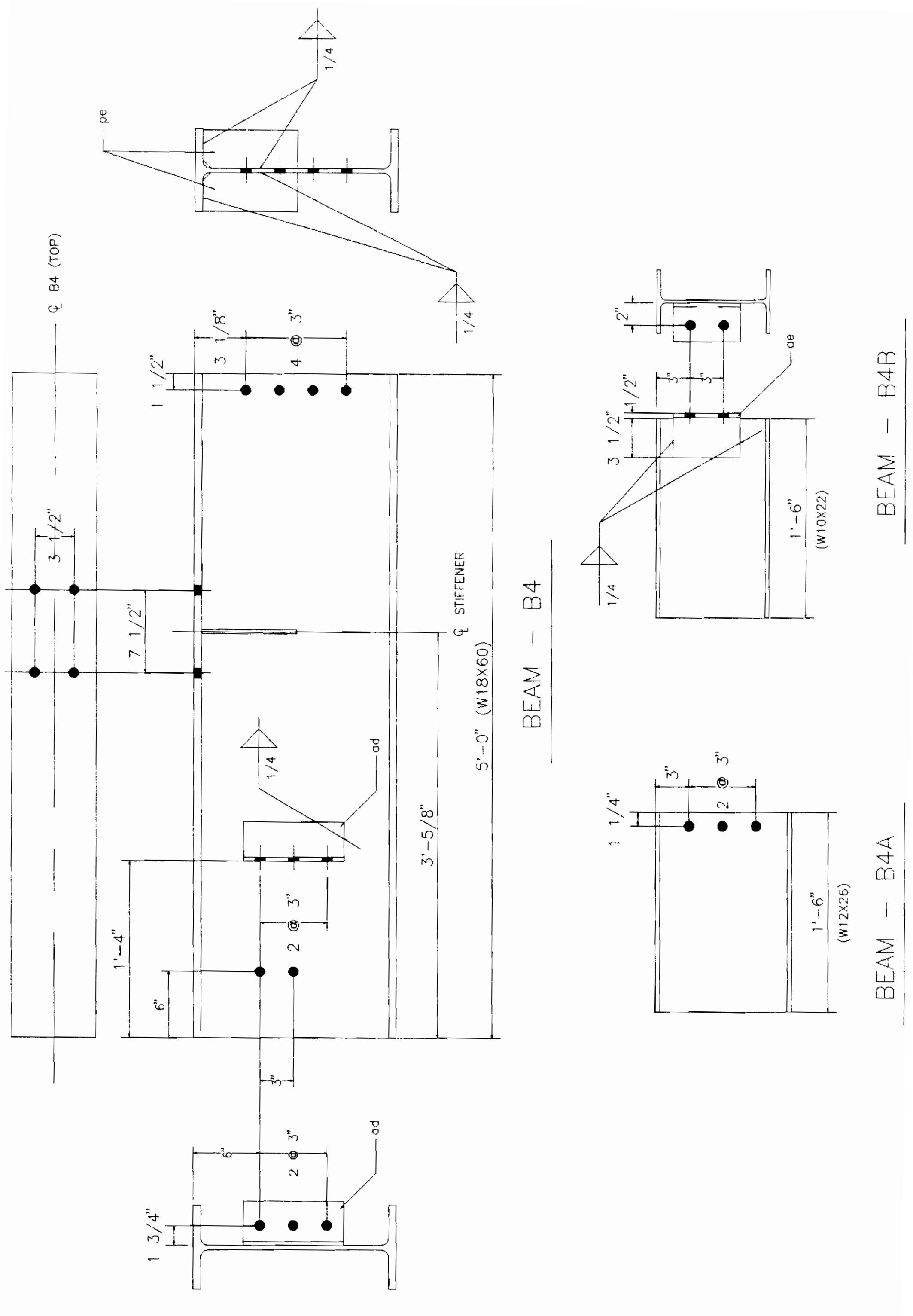

BEAM - B3
BEAM - B3A
BEAM - B3B
SHOP ASSEMBLED
(W21X44)
(W21X44)
(W12X50)
(W14X48)

Figure D.14



BEAM - B4
BEAM - B4A
BEAM - B4B
℄ B4 (TOP)
℄ STIFFENER
5'-0" (W18X60)
3'-5/8"
1'-4"
1'-6" (W12X26)
1'-6" (W10X22)
4 @ 3"
2 @ 3"
7 1/2"
3 1/2"
1 1/2"
1 1/4"
1 3/4"
3 1/8"
1/4
ad
ae
pe

Figure D.15

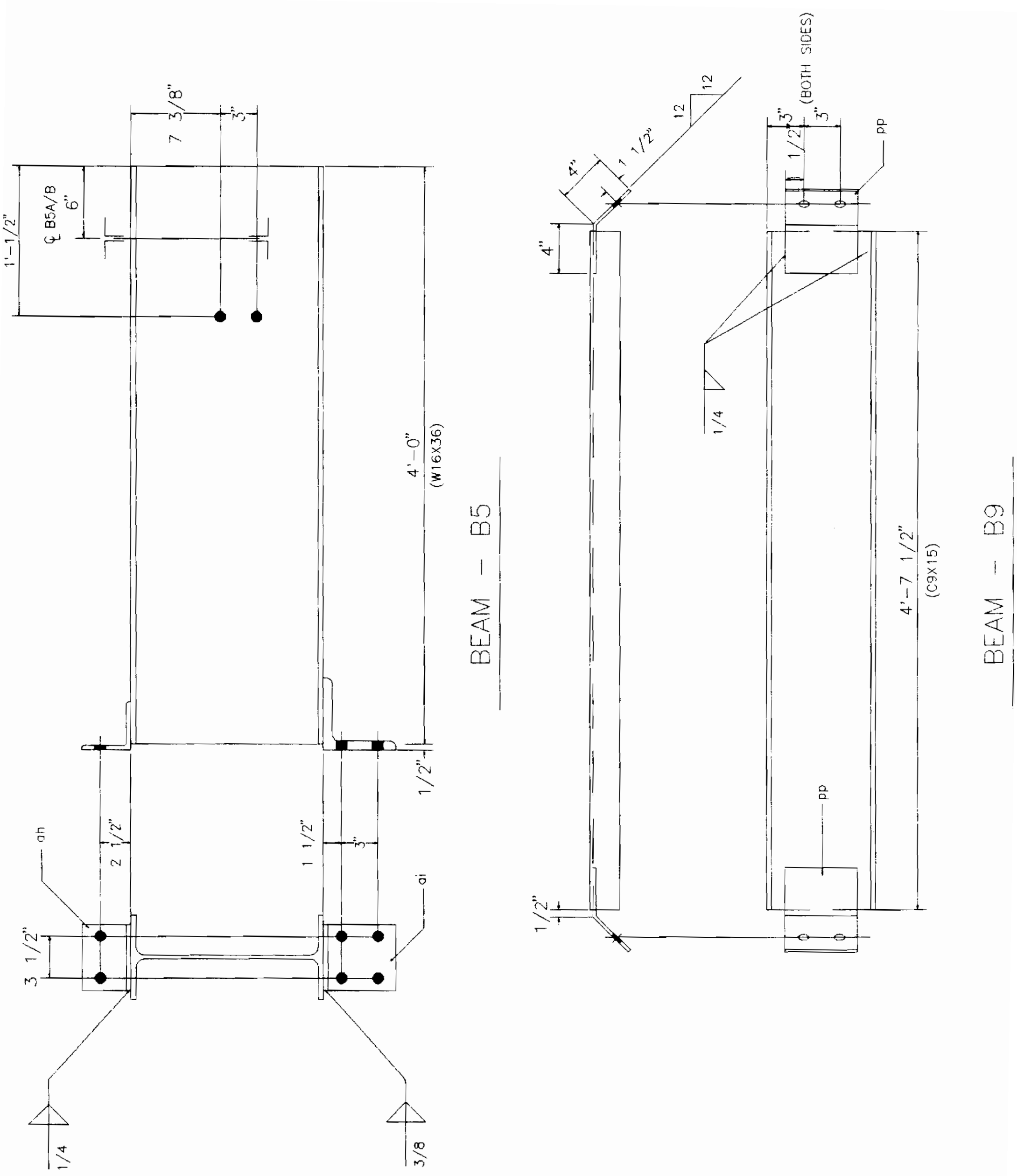
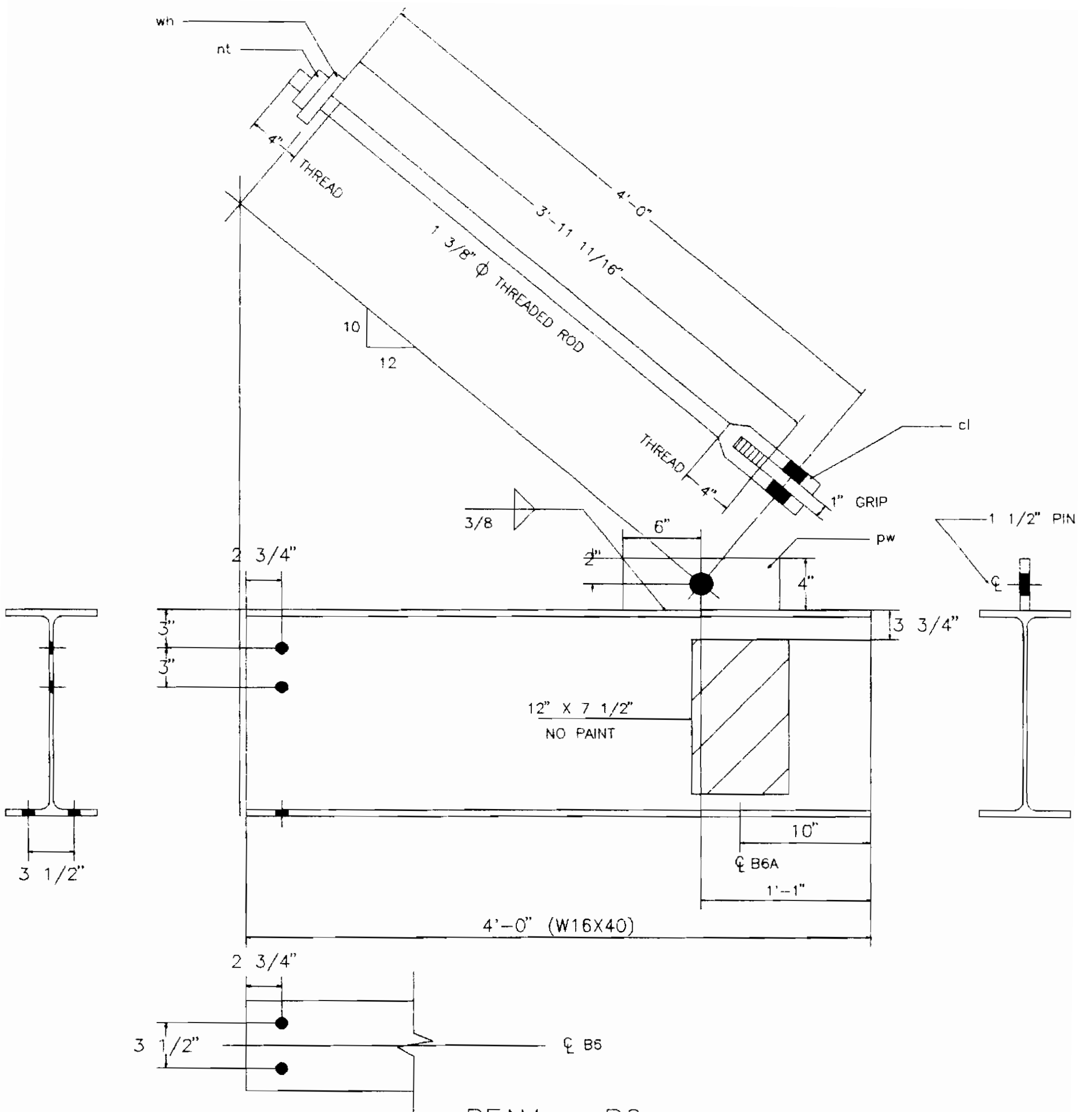
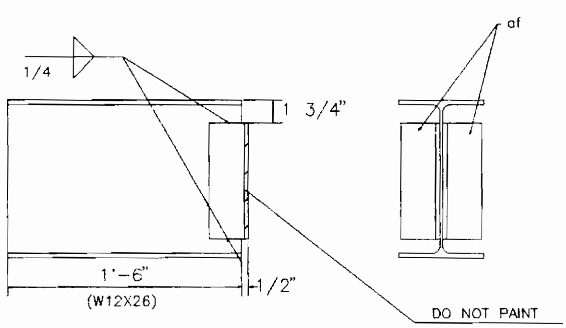

BEAM - B5
4'-0"
(W16X36)
1'-1/2"
7 3/8"
6"
2 1/2"
1 1/2"
3 1/2"
1/2"
1/4
3/8
ah
ai
BEAM - B9
4'-7 1/2"
(C9X15)
(BOTH SIDES)
12
12
4"
1/2"
pp
1/4

Figure D.16



BEAM – B6



BEAM – B6A

Figure D.17

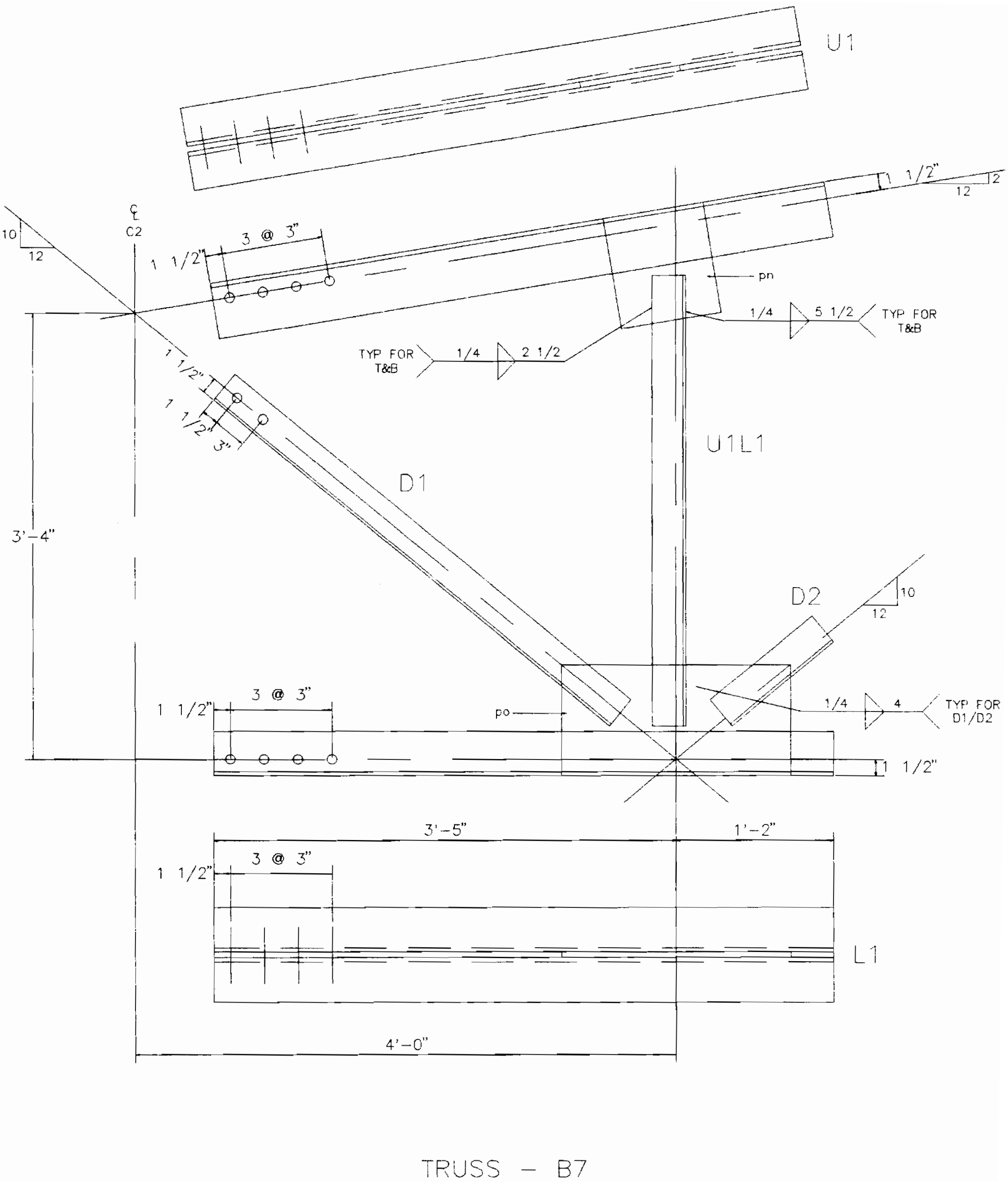

U1
1 1/2"
12
2
℄
C2
10
12
3 @ 3"
1 1/2"
pn
1/4
5 1/2
TYP FOR
T&B
TYP FOR
T&B
1/4
2 1/2
1 1/2"
1 1/2"
3"
U1L1
D1
3'-4"
D2
10
12
3 @ 3"
1 1/2"
po
1/4
4
TYP FOR
D1/D2
1 1/2"
3'-5"
1'-2"
3 @ 3"
1 1/2"
L1
4'-0"
TRUSS - B7

Figure D.18

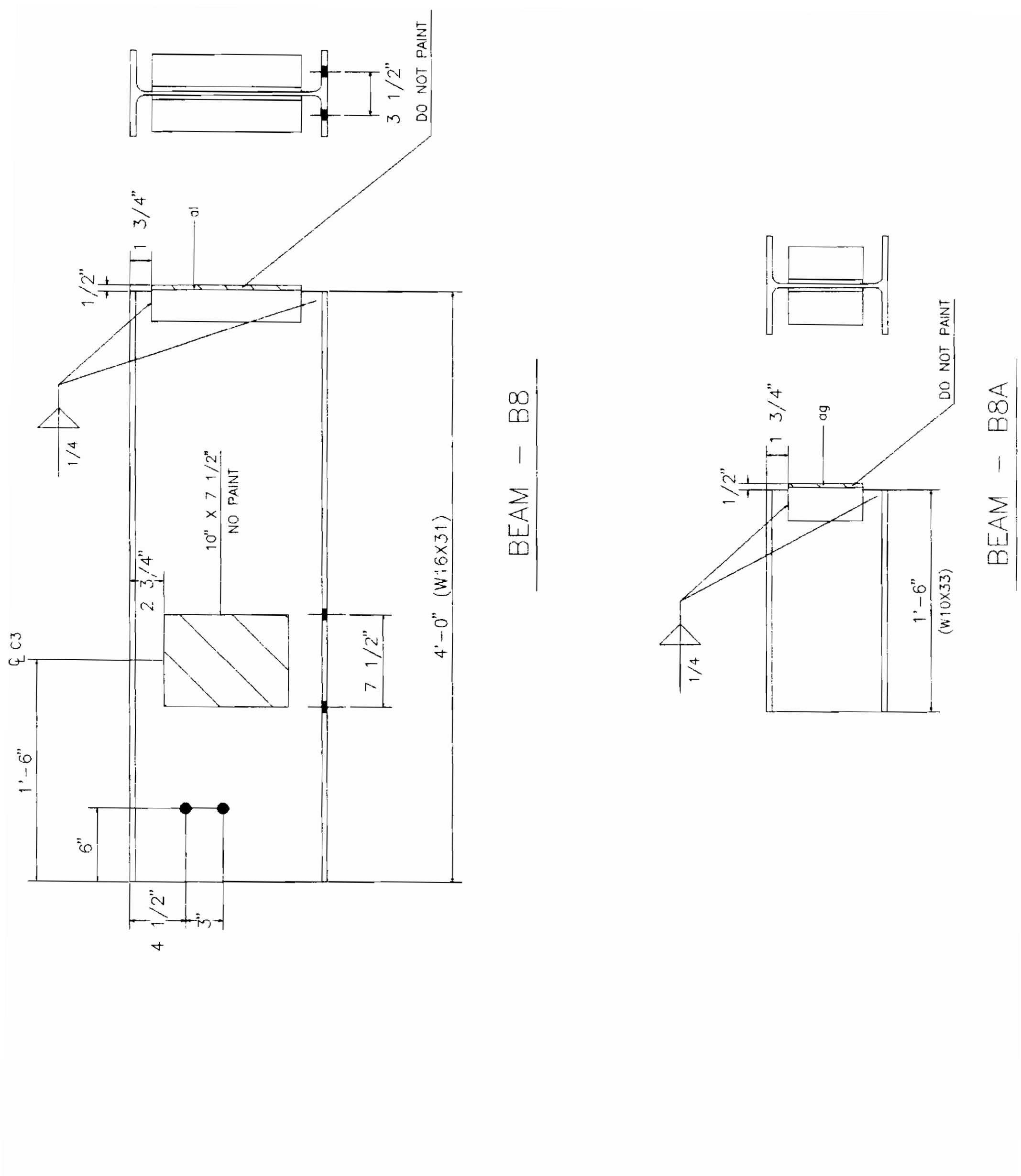

BEAM - B8
4'-0" (W16X31)
10" X 7 1/2"
NO PAINT
DO NOT PAINT
3 1/2"
7 1/2"
1'-6"
1/4
1 3/4"
1/2"
BEAM - B8A
1'-6"
(W10X33)
DO NOT PAINT
1/4
1 3/4"
1/2"

Figure D.19

APPENDIX E
NOMENCLATURE

a = length of rupture for block shear failure of a welded coped beam connection

a = distance between connectors in a built-up connection

a = distance from bolt line to application of prying force

A_b = bolt area

A_e = effective net area

A_{equiv} = equivalent area due to reduction

A_f = area of beam flange

A_{fe} = effective tension flange area

A_{fg} = gross flange area

A_{fn} = net flange area

A_g = gross area

A_{hole} = hole area = $(d_b + 1/16)t$

A_n = net area

A_{nt} = net tension area

A_{nv} = net shear area

A_{gt} = gross tension area

A_{gv} = gross shear area

A_{st} = stiffener area

A_w = area of beam web

A_{weld} = weld area

A_1 = area of steel bearing concentrically on concrete support

A_2 = total cross-sectional area of concrete support

b_f = flange width

b_{fc} = column flange width

b_p	=	plate width
b_s	=	stiffener width
B	=	width of base plate
c	=	length of cope
C	=	compressive force
$C_{a,b}$	=	coefficients used in extended end-plate connection design
C_w	=	warping constant
d	=	depth of member
d'	=	width of bolt hole parallel to tee stem or angle leg
d_b	=	bolt diameter
d_c	=	distance between fillets = d - 2k or T
d_c^b	=	bottom cope depth
d_c^t	=	top cope depth
d_h	=	hole diameter = $(d_b + 1/16)$
$d_{1,2}$	=	depth of respective beam member when considering column panel zone
D	=	number of sixteenths of an inch in fillet weld size
e	=	eccentricity or bolt edge distance
E	=	modulus of elasticity for steel = 29,000 ksi
f	=	computed stress or factor for computing coped beam strength
f_b	=	computed bending stress
f_c'	=	compressive strength of concrete at 28 days
f_d	=	factor for computing coped beam strength
f_p	=	computed compressive stress
f_t	=	computed tensile stress
f_v	=	computed shear stress
f_{vw}	=	computed weld shear stress
F	=	force or allowable stress

F_{cr}	=	critical yield stress
F_e	=	elastic buckling stress
F_{ex}	=	elastic flexural buckling stress about the major axis
F_{ey}	=	elastic flexural buckling stress about the minor axis
F_{ez}	=	elastic torsional buckling stress
F_{fu}	=	flange force
F_u	=	tensile stress
F_{uexx}	=	weld strength classification
F_y	=	yield stress
F_{yc}	=	column yield stress
F_{yf}	=	flange yield stress
F_{yst}	=	stiffener yield stress
F_{yw}	=	web yield stress
F.S.	=	factor of safety
g	=	bolt gage
G	=	shear modulus of elasticity for steel = 11,200 ksi
h	=	stiffener height
h_o	=	height of beam web between coped sections
H	=	flexural constant
$I_{x,y}$	=	moment of inertia about x and y-axis respectively
J	=	torsional constant
k	=	distance from outer face of flange to web toe fillet
k	=	equivalent length factor for compression members
k	=	factor for computing coped beam strength
kL	=	equivalent length
kL/r	=	slenderness ratio
l	=	length of yield for block shear failure of a welded coped beam connection

L	=	length of connection in direction of loading
L_{weld}	=	length of weld
m	=	number of shear planes in a fastener
m	=	cantilever dimension of base plate
M_{eu}	=	required flexural strength for extended end-plate connections
M_p	=	plastic moment capacity
$M_{p\ reduced}$	=	reduced moment capacity due to holes in member tension flange
$M_{1,2}$	=	respective applied moment when considering column panel zone
n	=	cantilever dimension of base plate
N	=	length of bearing
N	=	length of base plate
N_b	=	number of bolts in a joint
N_s	=	number of slip planes
p	=	length of supporting parallel to stem or leg of hanger tributary to each bolt in determining prying action
P_{bf}	=	concentrated force a column will sustain without stiffeners
p_e	=	effective span used to compute M_{eu} for extended end-plate connections extended end-plate connections
p_e	=	distance from centerline of bolt to nearer surface of tension flange in extended end-plate connections
P_n	=	nominal axial strength
P_u	=	required axial strength due to factored loads
r	=	governing radius of gyration
r_v	=	nominal shear value for one fastener
$r_{x,y}$	=	radius of gyration about a and y axes respectively
R_n	=	nominal strength
R_u	=	required strength due to factored loads
s	=	bolt spacing

S_{pl}	=	section modulus of a plate
S_{tee}	=	section modulus of a tee
t	=	thickness
t_f	=	flange thickness
t_{fb}	=	beam flange thickness
t_p	=	plate thickness
t_s	=	stiffener thickness or tee stem thickness
t_w	=	web thickness
t_{wc}	=	column web thickness
t_{weld}	=	fillet weld size
T	=	tensile force due to service loads
T_b	=	minimum pretension of a high-strength bolt
T_E	=	tensile force due to earthquake load
T_L	=	tensile force due to live load
T_m	=	minimum fastener tension
T_n	=	nominal tensile strength
T_u	=	required tensile strength due to factored loads
T_w	=	tensile force due to wind load
U	=	reduction coefficient, used in calculating effective net area $= 1 - \frac{\bar{x}}{L}$
V	=	shear produced by factored loading
V_n	=	nominal shear strength
V_s	=	story shear
V_u	=	required shear strength
w	=	weld size
w_e	=	effective width according to Whitmore model
$\bar{x}$	=	connection eccentricity and distance to centroid along respective axis

x_o, y_o	=	coordinates of the shear center with respect to the centroid
Z_x	=	plastic section modulus
α	=	ratio of moment at bolt line to moment at stem line for determining prying action
α_m	=	coefficient used in calculating the design moment, M_{eu}, for end plate connections
δ	=	ratio of net area (at bolt line) and gross area (at face of stem or angle leg) used in determining prying action
ϕ	=	resistance factor
λ_e	=	equivalent slenderness parameter
λ_r	=	limiting slenderness parameter or non-compact member
μ	=	mean slip coefficient
π	=	pi
υ	=	poisson's ratio

VITA

Robert A. Kerr was born in Colorado Springs, Colorado on August 15, 1970. He graduated from South Lakes High School in Reston, Virginia in 1988, and received his Bachelor of Science in Civil Engineering from Virginia Polytechnic Institute and State University in December 1992. He entered the structures graduate program of the Charles E. Via Department of Civil Engineering in the spring of 1993 to pursue a Master of Engineering in Civil Engineering.

Robert A. Kerr