Design of Fully Restrained Moment Connections AISC LRFD 3rd Edition (2001)

COURSE CONTENT

1. TYPES OF CONSTRUCTION

In steel framework, beam end connections occur quite often that they influence costs very strongly and have attracted a great deal of attention from design engineers and researches. This effort has resulted in a great variety of forms that can be executed safely. The study of beam end connections entails the considerations of a range of assumptions made in frame analysis regarding these connections. The new AISC specification provides for three basic types of framing, which relate to the end connections of beams to columns.

Section A2.2 of the LRFD specification defines the following types of construction:

Type FR (fully restrained), which is commonly referred as "rigid frame" (continuous frame), considers that connections have enough stiffness to maintain the angles between the connected members. In other words, a full transfer of moment and little or no relative rotation of members within the joint. This type of connection was formerly referred to as Type 1 construction in previous editions of the AISC.

Type PR (partially restrained) assumes that the connections have insufficient stiffness to maintain the angles between the intersecting members. This type of connections requires that the strength, stiffness and ductility characteristics of the connections be considered in the analysis and design. This course will not cover this type of connection, formerly referred to as Type 3 construction in previous editions of the AISC. Part 11 of the LRFD deals with an alternative and a more simplified approach, namely the "*flexible moment connection*".

Simple framing is the other type of construction, where the connection restraint is ignored (unrestrained, free-ended), and the connection is designed to resist gravity loads only while allowing relative rotation of the connected members (no moment transfer is taken into account). This type of connection was formerly referred to as Type 2 construction in previous editions of the AISC.

The general behavior of these three types of connection is illustrated below in Figure No.1; typical examples of these connection types are shown on Figures 2, 3, and 4.

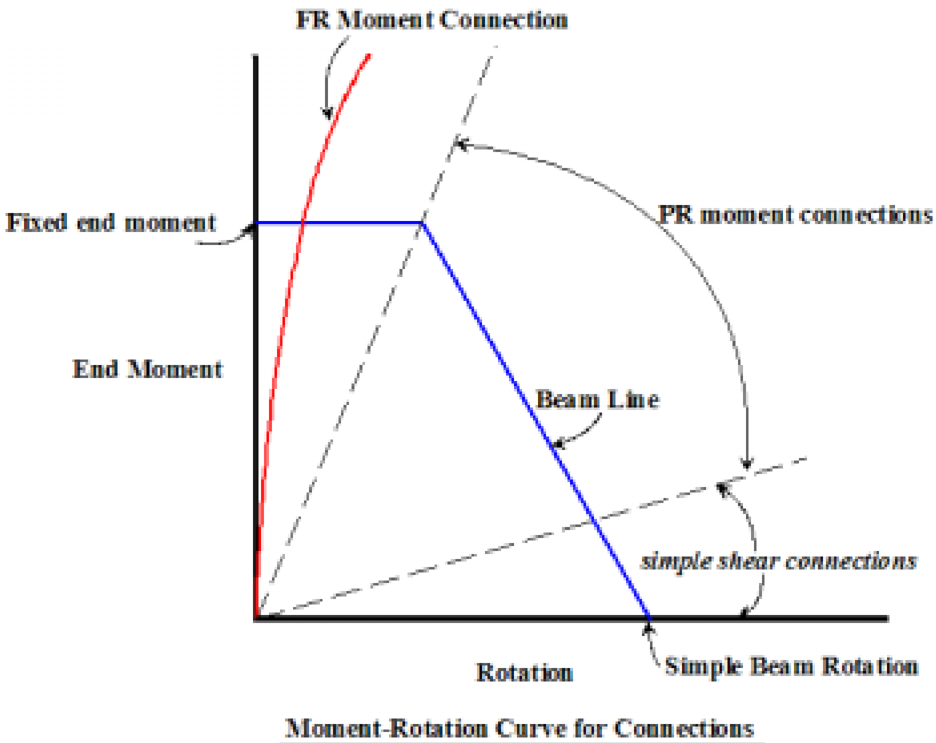
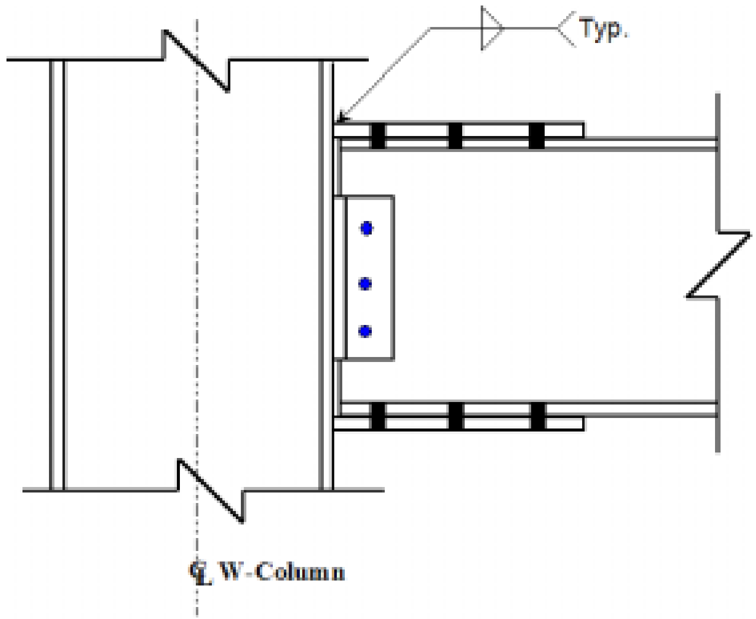
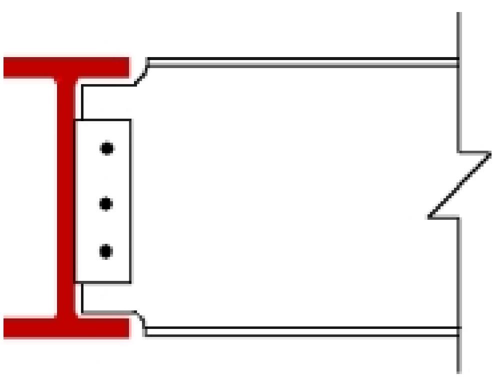


Figure No.1



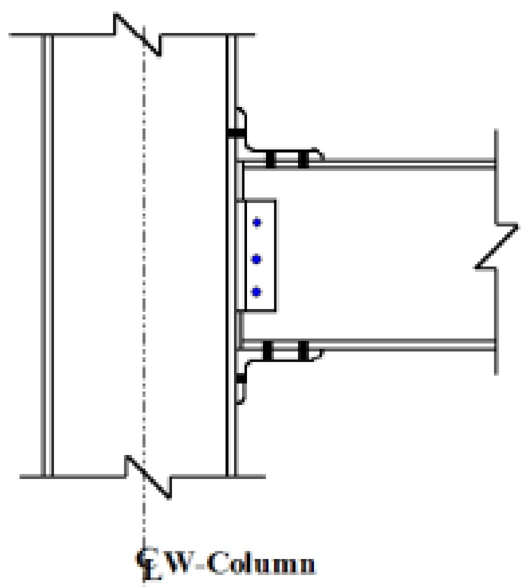
Flanged-Plated FR Moment Connection

Figure 2



Simple Framing Connection

Figure 3



Flange-Angle Flexible Moment Connection

Figure 4

1. AISC LRFD 3rd Edition – November 2001

Load and resistance factor design (LRFD) is based on a consideration of failure conditions rather than working load conditions. Members and its connections are selected by using the criterion that the structure will fail at loads substantially higher than the working loads. Failure means either collapse or extremely large deformations.

Load factors are applied to the service loads, and members with their connections are designed with enough strength to resist the factored loads. Furthermore, the theoretical strength of the element is reduced by the application of a resistance factor.

The equation format for the LRFD method is stated as:

$$\Sigma\gamma_i Q_i = \phi R_n \qquad \textbf{(Eq. 1)}$$

Where:
Q_i = a load (force or moment)

γ_i = a load factor (LRFD section A4 Part 16, Specification)

R_n = the nominal resistance, or strength, of the component under consideration

ϕ = resistance factor (for bolts and welds given in LRFD Chapter J, Part 16)

The LRFD manual also provides extensive information and design tables for the design considerations of bolts in Part 7, Part 9, 10 and Part 16 Chapter J, section J3. Design considerations for welds are addressed in Part 8, and Part 16, Chapter J, section J2.

Other parts of the manual cover connections such as flexible moment connections (Part11), bracing and truss connections (Part 13), column splices (Part 14), hanger connections, bracket plates, and crane-rail connections (Part 15). Our discussion will be limited to the design of fully restrained (FR) moment connections presented in Part 12 and Part 16, Chapter J.

3. Basic Behavior of FR Moment Connections

Fully restrained (FR) moment connections must have sufficient rigidity to maintain the angles between the intersecting angles as shown on Fig. No. 5. Since it is quite difficult if not impossible to achieve full rigidity in a FR moment connection, the small amount of flexibility present is usually neglected and the connections are idealized as preventing relative rotation.

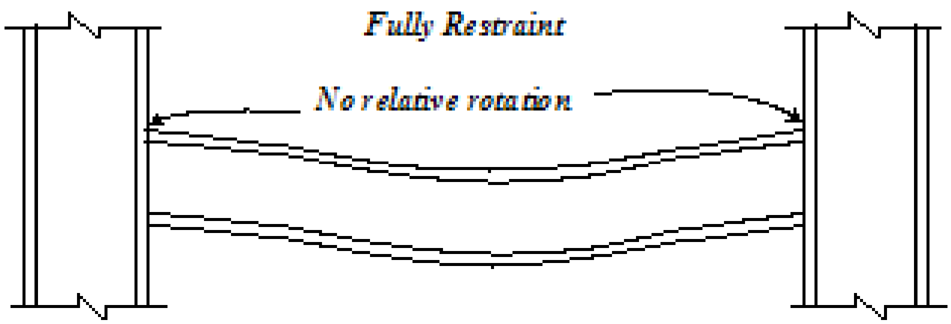


Figure 5

LRFD specification Section B9 states that end connections in FR construction must be designed to carry the factored forces and moments, allowing for some inelastic but self-limiting deformation of a part of the connection.

In a FR connection the moment can be resolved into an effective tension-compression couple acting as axial forces at the beam flanges, as shown on Fig. No. 6, while the shear force is considered to be resisted entirely through the web shear connection. The eccentricity in the shear connection can be neglected entirely since the angle between the members in a FR moment connection remains unchanged under loading. Axial forces are normally assumed to be distributed uniformly across the beam flanges, and are added to the couple forces from the applied moment.

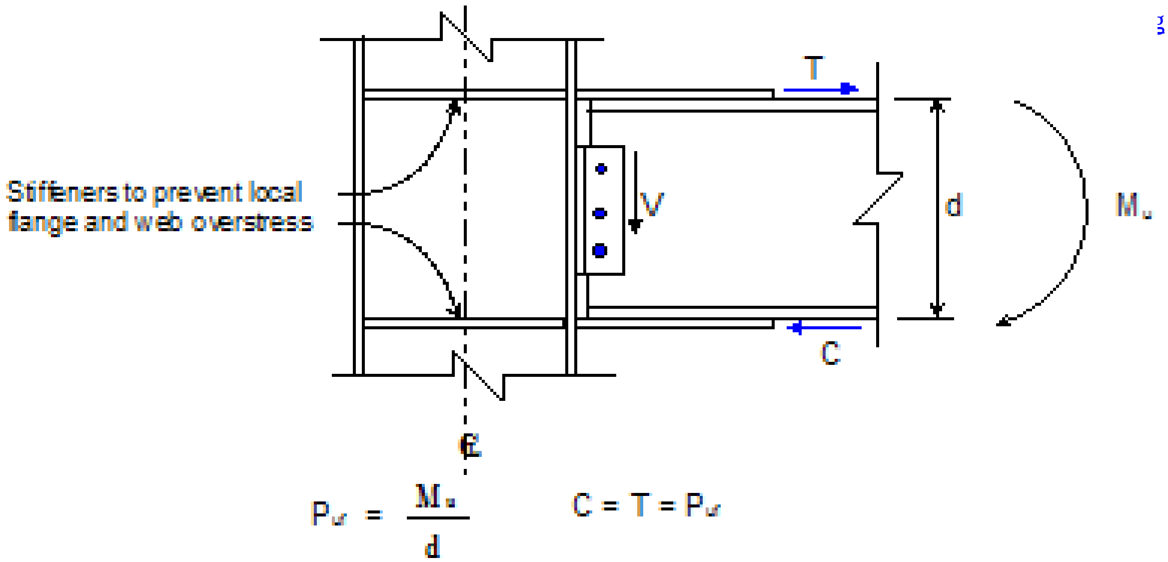


Figure 6

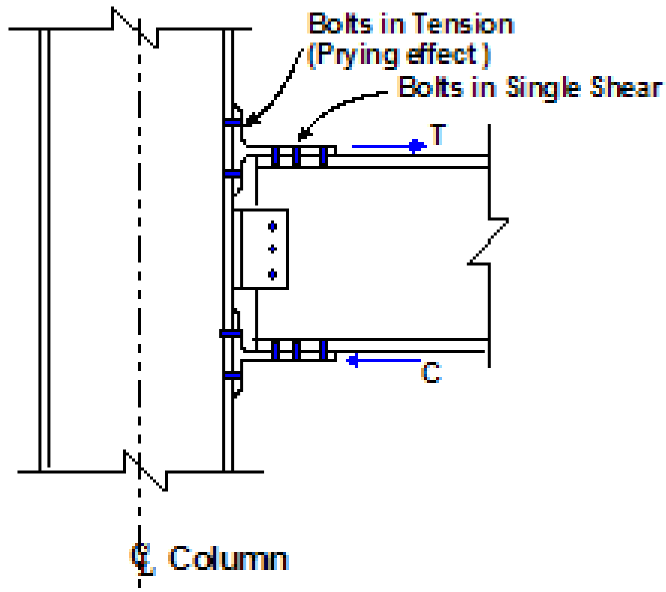
Furthermore, moment connections transmit concentrated forces to column flanges, and these forces must be accounted in the design to prevent local flange bending (from the tension force), local web yielding, web crippling, and web compression buckling caused by the compression force. Horizontal stiffeners may be required to address these local effects in accordance with LRFD Specification (Part 16), Chapter K, sections K1.2, K1.3, K1.4, and K1.6 respectively.

There are a great variety of arrangements for FR moment connections and we will concentrate on three major designs: a) the flange Tee-Stub bolted FR moment connection, b) the flanged-plated FR moment connection, and c) the directly welded flanged FR moment connections. Both bolted and welded considerations will be covered for these connections.

4. Bolting Considerations for FR Moment Connections

The LRFD allows bolts in bearing in either standard holes or slotted holes perpendicular to the line of force. The applicable limit states for the bolts are covered under Part 7 of the LRFD.

Moment resistance of bolted FR moment connections depends on tension and shear in the fasteners. One of the most common bolted FR moment connections is the Flange Tee-Stub connection shown on Fig. No.7.



Split-Beam Tee Bolted Moment Connection

Figure 7

The design of this connection involves the transfer of the tensile force T through bolts (in the stem of tee) in single shear, and direct tension through bolts in the flange of the tee. The bolts in tension require consideration of "prying action".

Prying forces arises when a relatively thin plate deflects outward, thus pressing the unsupported edges against the supporting piece, see Fig. 8.

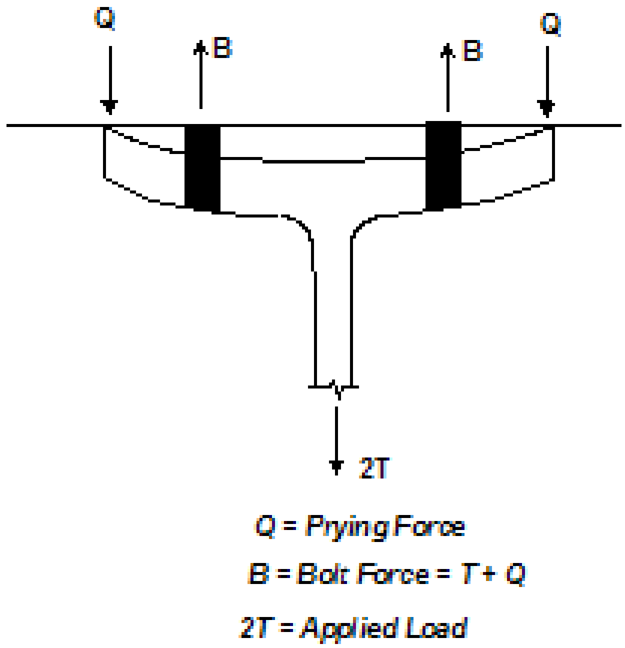


Figure 8

LRFD section J3.6 states that for bolts “the applied load shall be the sum of the factored loads and any tension resulting from prying action produced by deformation of the connected parts”. Procedure to compute prying forces is covered in Part 9 of the LRFD.

For FR moment bolted connections the most commonly used high strength bolts come in two grades: ASTM A325 and ASTM 490. These bolts can be used in several joint types, such as snug-tightened (bearing joints), pretensioned joints, and slip-critical joints. Each joint type is to be specified in accordance with the required performance in the structural connection.

Although the unfinished bolt designated as ASTM A307 is not precluded to be used in FR moment connections (except for the limitations specified in LRFD J1.11), these bolts are primarily used in light structures, secondary or bracing members, platforms, catwalks, purlins, girts, light trusses, and other structures with small loads and static in nature. The A307 bolts are used predominantly in connections for wood structures.

Example 1

Design a bolted T-Stub moment connection for the beam shown on Figure 9, supporting both gravity and wind loads.

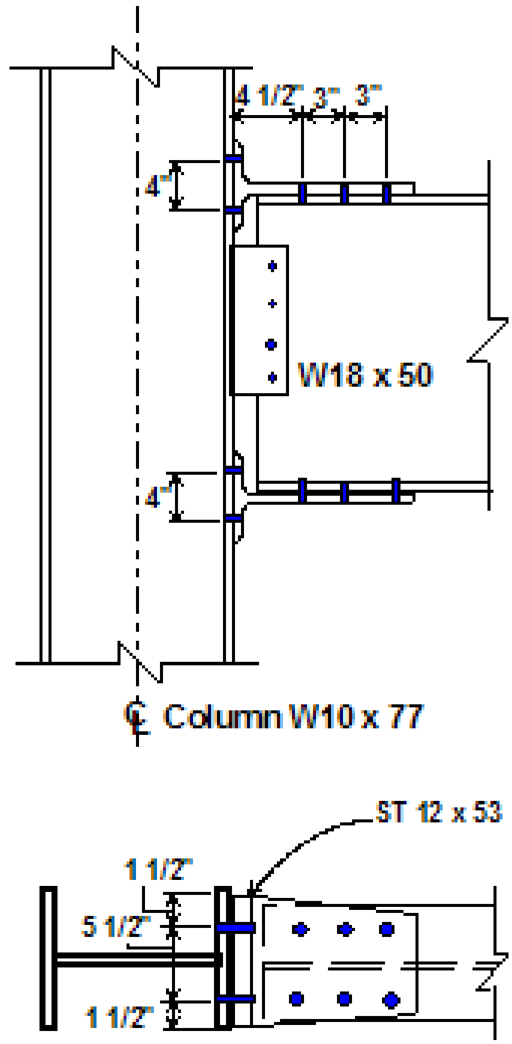


Figure 9

Design Parameters:
Steel Specification ASTM A992 - Bolts ASTM A325
Loads:
Dead Moment, M_D = 20 ft-kips
Live Moment, M_L = 38 ft-kips
Wind Moment, M_w = 82 ft-kips
Shear to be resisted by web connection (not in the scope of this course)

Step1: Determine the factored design moment:

1.4 D = M = 28 ft-k
1.2D + 1.6L = M = 84.8 ft-k
1.2D + 1.6W + 0.5 L = M = 174.2 ft-k ←----- Governs

(Note, other combinations may be considered in an actual design, refer to ASCE 7-02)

For a W18 x 50, d = 18 in.
Step 2 : Find the maximum Tension force on the flange

T = C = M / d = 174.2 x 12 / 18 = 116.1 kips

Step 3: Bolt requirements

Try 7/8" diameter bolts – Single shear at the beam flanges and tension at the column face

a) Tee to beam flanges:

From AISC LRFD Table 7-10, the design shear strength of a 7/8" ϕ A325 bolt with the threads included in the shear plane (snugged-tight joint)
ϕF_V = 21.6 kips
[from $\phi F_n A_b$ (Table J3.2, where $\phi = 0.75$, F_n = 48 ksi, and A_b = 0.601 in^2)]

No. of bolts required = 116.1 / 21.6 = 5.4 USE 6 bolts to connect the T-stubs to the beam flanges

b) Tee to column:

From AISC LRFD Table 7-14, the design tensile strength of a 7/8" ϕ A325 bolt is given as:
ϕF_t = 40.6 kips
[from $\phi F_n A_b$ (Table J3.2, where $\phi = 0.75$, F_n = 90 ksi, and A_b = 0.601 in^2)]

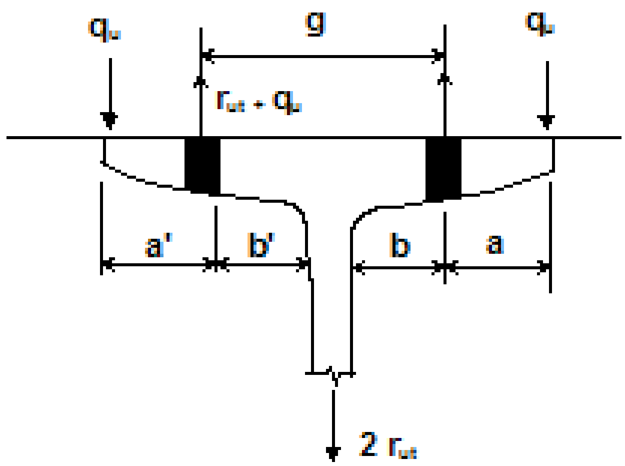
No. of bolts required = 116.1 / 40.6 = 2.9 USE 4 bolts to connect the T-stubs to the column flange

Check Prying effect, Figure 10 (from LRFD Part 9):

T-Stub Properties: t_f = 1.09 in t_w = 0.62 in b_f = 7.87 in g = 4 in

The thickness required to eliminate prying action t_{min} is determined by:

$$t_{min} = \sqrt{\frac{4.44\, r_{ut}\, b'}{p\, F_y}} \qquad \text{(Eq. 2)}$$



r_{ut} = required strength
(factored tension per bolt)

$$b' = \left[b - \frac{d_b}{2} \right]$$

d_b = bolt diameter, in

<u>Figure 10</u>

Where:
r_{ut} = 116.1 / 4 = 29 kips
p = tributary length of flange per pair of bolts
p = (5.5 + 2 x 1.5) / 2 = 4.25” <= g then p = 4 in (see Figure No. 9)
b = 4/2 – 0.62/2 = 1.69 in
b’ = 1.69 – 0.875/2 = 1.25 in
F_y = 36 ksi (T-stub)

Then, $t_{min.}$ = 1.06 in. < t_f = 1.09 in. Therefore prying effect is eliminated

USE 4 – 7/8” ϕ A325 bolts to connect the T-stub to the column-

Step 4: Check T-stub Capacity

a) Check the capacity of the web in tension

From LRFD Specification part D:
Yielding of the gross section; $\phi_t P_n = 0.90 \times F_y \times A_g$

F_y = Specified minimum yield strength of the T, 36 ksi
Ag = gross area of the member

$\phi_t P_n$ = 116.1 kips, then the minimum width required is

w_{min} = 116.1 / (0.9 x 36 x 0.62) = 5.78 in

For rupture in the net section:

$\phi_t P_n = 0.75 \times F_u \times A_e$
F_u = Specified minimum tensile strength of T-stub, 58 ksi
A_e = Effective net area of member

Minimum A_e required = 116.1 / (0.75 x 58) = 2.67 in^2

The minimum effective width b_e = 2.67 / 0.62 = 4.3 in

The provided width of the T-stub is at a minimum 7 ½ in. (the flange width of the W18 x 50), and taken as 8 inches at the column face,
$b_{emin} = 7.5 – 2(1) = 5.5$ in > 4.3 in

The capacity of the T-stub web is considered adequate in resisting the tensile forces.

b) The capacity of the T-stub flange at the column face
The flange thickness is larger than the required t_{min} for prying forces, so the flange of the T-section is thick enough to resist any local bending thus reducing any prying action to insignificant level.

Step 5: Other Considerations for the Design of a FR Moment Connection

a) For the configuration and final connection evaluation the followings items will require further investigation that are beyond the scope of this course such as: geometric layouts of structural bolts such as, size and use of bolt holes, minimum and maximum bolt spacing, minimum and maximum edge distance, bearing strength of bolts, and the design rupture strength of the connected parts.

These limit states has been addressed in PDH course S-134,
Design of Bolts in Shear-Bearing Connections per AISC LRFD 3rd Edition (2001)

b) Comments on the fabrication of FR moment connection

i) The beam end must be cut back far enough to keep the flange thickness from interfering with the bolt head.
ii) The nuts should be placed on the inside of the column flange
iii) The beam web connection (to resist the vertical shear force) must clear the edge of the T-stub flange
iv) Shims should be furnished in thicknesses of 1/16-in. to 1/8-in. to allow for beam and fabrication tolerances.

Finally, the main advantage of the T-stub bolted connection is the provision of a bolted connection that eliminates all complications that may arise in field welded connections. The extended end-plate FR moment connection is another field bolted connection that avoid field welding. This connection is covered in depth by the AISC in Design Guide number 13.

5. Column Limit States

Columns in FR moment connections must be checked for the following limit states conditions (see Figure 11):

Flange local bending (LRFD Specification Part 16, Section K1.2)

Web local yielding (LRFD Specification Part 16, Section K1.3)

Web crippling (LRFD Specification Part 16, Section K1.4)

Web compression buckling (LRFD Specification Part 16, Section K1.6)

Web panel-zone shear (LRFD Specification Part 16, Section K1.7)

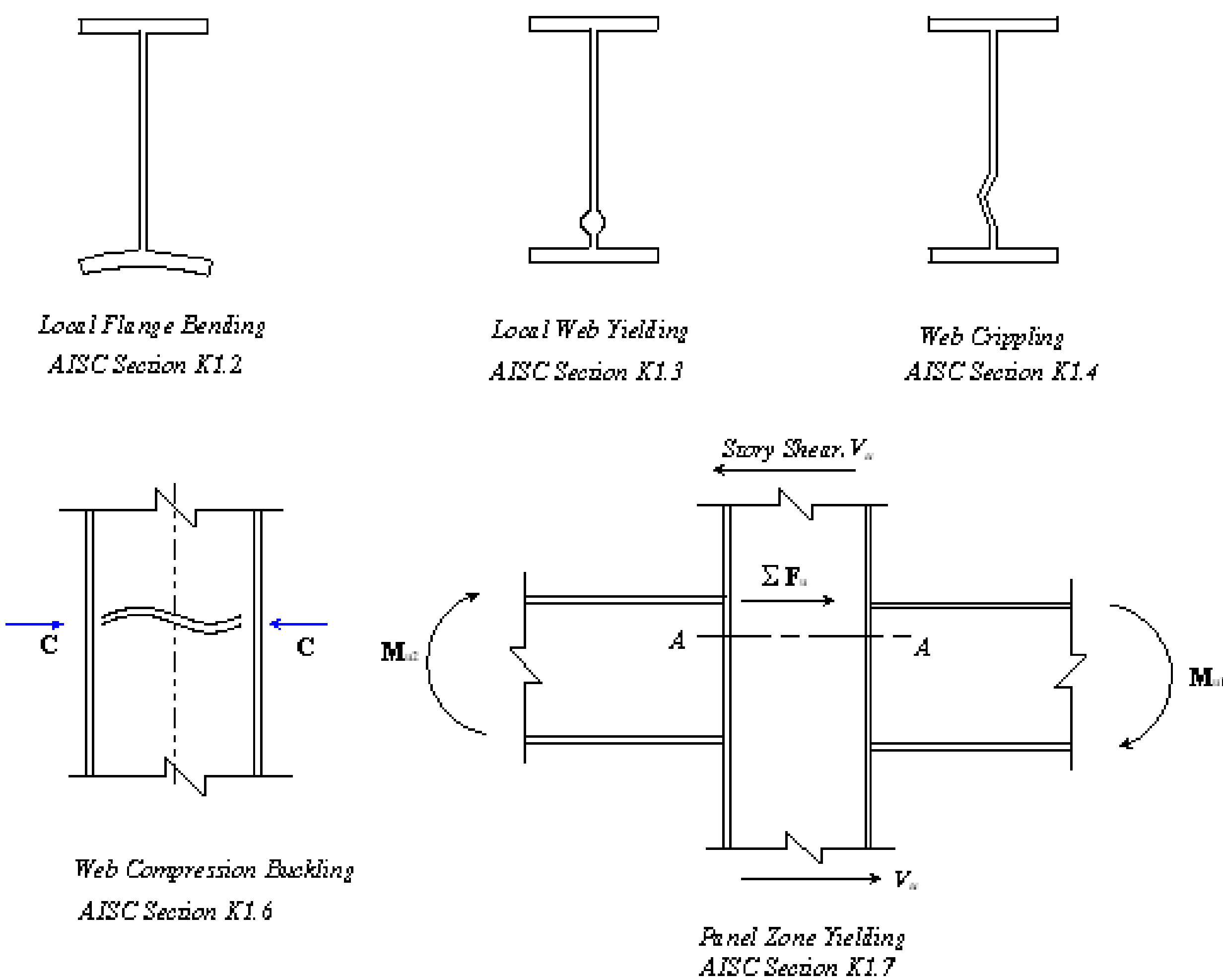


Figure 11

5.a) Flange Local Bending, LRFD Chapter K, section K1.2

Applicable to both tensile single-concentrated forces and the tensile component of double-concentrated forces, see Figure No. 12.

A pair of transverse stiffeners extending at least one-half the depth of the web is required when the required strength of the flange exceeds ϕR_n as given by LRFD equation (K1-1):

Where:
$\phi = 0.90$ and $R_n = 6.25\, t_f^2\, F_{yf}$ **LRFD Equation (K1-1)**

F_{yf} = yield stress of the column flange, ksi
t_f = thickness of the loaded column flange, in.

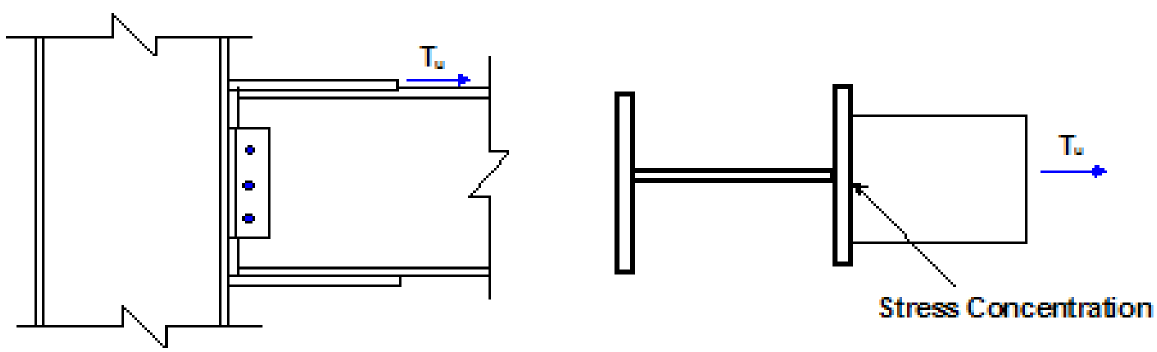


Figure 12

From example 1, $T_u = 116.1$ kips and $t_f = 0.87$ in. for W10x77 column

$\phi R_n = 0.90 \times 6.25 \times (0.87)^2 \times 50 = 212.88 \text{ k} > 116.1 \text{ k}$ ∴OK no stiffeners are req'd

5.b) Column Web Local Yielding, LRFD Chapter K, section K1.3

Applicable to both tensile or compressive single-concentrated forces and both components of double-concentrated forces, see Figure No. 13.

Either a pair of transverse stiffeners or a doubler plate, extending at least one-half the depth of the web is required when the required strength of the web at the toe of the fillet exceeds ϕR_n as given by LRFD equation (K1-2):

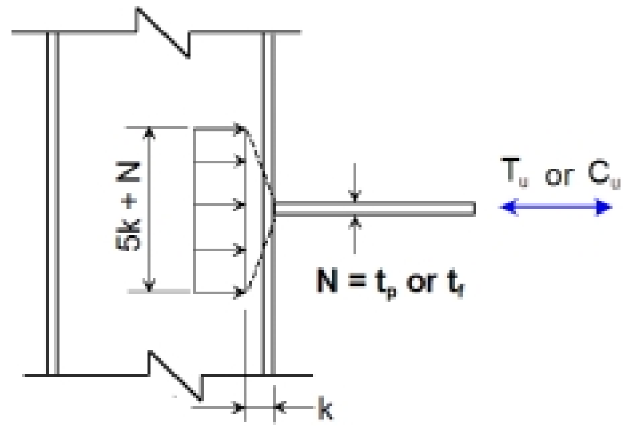


Figure 13

Where:

$\boldsymbol{\phi = 1.0}$ **and** $\mathbf{R_n = (5k + N)\, F_{yw}\, t_w}$ **LRFD Equation (K1-2)**

F_{yw} = yield stress of the column web, ksi
t_w = web thickness of loaded column, in.
k = distance from outer face of the flange of the web toe of the fillet, in
N = length of bearing, beam flange thickness or plate thickness, in.

From example 1, $T_u = C_u$ = 116.1 kips, k = 1.37 in. and t_w = 0.53 in. for W10x77 column
N = 0.62 in. (ST 12 x 53 web thickness), considering that the concentrated force is delivered through the ST web

ϕR_n = 1.0 x (5 x 1.37 + 0.62) x 50 x 0.53 = 197.96 k > 116.1 k ∴OK no stiffneners are req'd

5.c) Column Web Crippling, LRFD Chapter K, section K1.4

Applicable to a compressive single-concentrated force and the compressive component of double-concentrated forces.

Either a transverse stiffener, a pair of transverse stiffeners, or a doubler plate, extending at least one-half the depth of the web is required adjacent to a concentrated compressive force when the required strength of the web exceeds ϕR_n as given by LRFD equation (K1-4):

Where:
$\phi = 0.75$

$$R_n = 0.80\, t_w^2 \left[1 + 3\,(N/d)\,(t_w/t_f)^{1.5}\right] \sqrt{\frac{E\, F_{yw}\, t_f}{t_w}} \qquad \text{LRFD Equation (K1-4)}$$

Where, all terms same as section 5.b) above and
d = overall depth of the column, in
t_f = column flange thickness, in

From example 1, C_u = 116.1 kips, d = 10.6 in. for W10x77 column
N = 0.62 in. (ST 12 x 53 web thickness).

$$\mathbf{R_n} = 0.80 \times 0.53^2 \left[1 + 3\,(0.62 / 10.6)\,(0.53 / 0.87)^{1.5}\right] \sqrt{\frac{29 \times 10^3 \times 50 \times 0.87}{0.53}} = 375.6 \text{ k}$$

ϕR_n = 0.75 x 375.6 = 281.7 k > 116.1 k ∴OK stiffeners are not req'd

5.d) Column Web Compression Buckling, LRFD Chapter K, section K1.6

This limit state is applicable when concentrated loads from beam flanges are applied to both column flanges. When only one column flange is subjected to a concentrated load, the overall web buckling limit state does not need to be checked.

Either a single transverse stiffener, or a pair of transverse stiffeners, or a doubler plate, extending the full depth of the web is required adjacent to a concentrated compressive forces at both flanges when the required strength of the web exceeds ϕR_n as given by LRFD equation (K1-8):

Where:
$\phi = 0.90$

$$R_n = \frac{24\, t_w^3 \sqrt{E\, F_{yw}}}{h} \qquad \text{LRFD Equation (K1-8)}$$

Where, all terms same as previously defined and
h = column web depth clear of fillets, d - 2 k, in

From example 1, C_u = 116.1 kips assumed acting on both sides of the column
h = 10.6 – 2(1.37) = 7.86 in. for W10x77 column

$$R_n = \frac{24 \times 0.53^3 \sqrt{29 \times 10^3 \times 50}}{7.86} = 547.4 \text{ k}$$

ϕR_n = 0.90 x 547.4 = 492.66 k > 116.1 k ∴OK stiffeners are not req'd

5.e) Column Web Panel-Zone Shear, LRFD Chapter K, section K1.7

Within the boundaries of a rigid connection of members whose webs are in a common plane, either doubler plates or diagonal stiffeners must be provided when the required strength exceeds ϕR_v as given by LRFD equations (K1-9 through K1-12).

Equations K1-9 and K1-10 apply when the effect of panel-zone deformation on frame stability is not considered in the analysis. This condition limits the panel-zone behavior to the elastic range, and may be considered appropriate for connections designed to resist wind loads.

Equations K1-11 and K1-12 apply when frame stability, including plastic panel-zone deformation is considered in the analysis. These equations recognize the additional inelastic shear strength available in a connection with adequate ductility. The inelastic shear strength is often used for the design of frames in high seismic zones, and should be used when the panel zone is to be designed to match the strength members from which it is formed.

This course will limit the discussion to equations K1-9 and K1-10.

Two conditions must be investigated when the effect of panel-zone deformation on frame stability is not considered in the analysis:

Where ϕ = 0.90 for both cases and R_v is computed as:

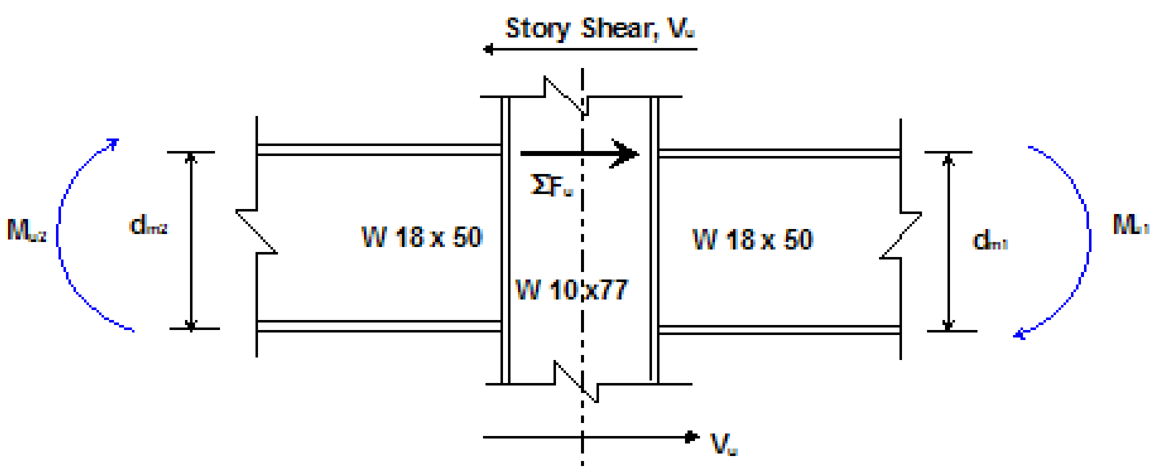
a) For $P_u \leq 0.4 P_y$

$R_v = 0.60 F_y d_c t_w$ LRFD Equation (K1-9)

b) For $P_u > 0.4 P_y$

$R_v = 0.60 F_y d_c t_w (1.4 – P_u / P_y)$ LRFD Equation (K1-10)

From example 1,



Forces in Panel Zone

Assume that $M_{u1} = M_{u2}$ = 174.2 ft-k, P_u = 560 kips (column axial load) and the calculated story shear V_u = 6.4 kips

The calculated factored shear force $\Sigma F_u = (M_{u1} / d_{m1}) + (M_{u2} / d_{m2}) – V_u$

Then ΣF_u = 116.1 + 116.1 – 6.4 = 225.8 kips

$P_y = A F_y$ = 22.6 x 50 = 1,130 kips for W10x77

Since P_u = 560 kips > 0.4 P_y = 452 kips, use equation K1-10

ϕR_v = 0.90 x 50 x 10.6 x 0.53 (1.4 – 560 / 1130) = 228.65 kips > 225.8 kips

∴OK stiffeners are not req'd

6. Flange-Plated FR Moment Connections

A flanged plated FR moment connection is made up of a web shear connection and top and bottom flange plates that connect the flanges of the supported beam to the supporting column. These flange plates are welded to the supporting columns and may be bolted or welded to the flanges of the supported beam, Figure No. 2. The shear is transferred to the columns via shear tab between the beam web and the column.

The design of the flange plates involves computing the strength required to resist the components of the moment both tension and compression, and the required strength of either the bolts or welds needed to attach to the top and bottom flange of the supported beam. If welding is used they are usually specified as fillet weld. Should the plates be bolted, then the effect of bolt holes, and block shear needs to be considered. The flange plates are groove welded or fillet welded to the columns to complete the load transfer. Usually these plates are shipped loose for field attachment to the column due to the potential assembly problems caused by mill tolerances in both the beam and the column.

Example 2

Design a moment-resisting connection for the W18 x 46 beam shown in Figure No. 14 with flange plates. Column size: W12 x 96.

Design Parameters:
Steel Specification ASTM A992 - Bolts ASTM A325
Factored end-moment: M_u = 180 ft-kips

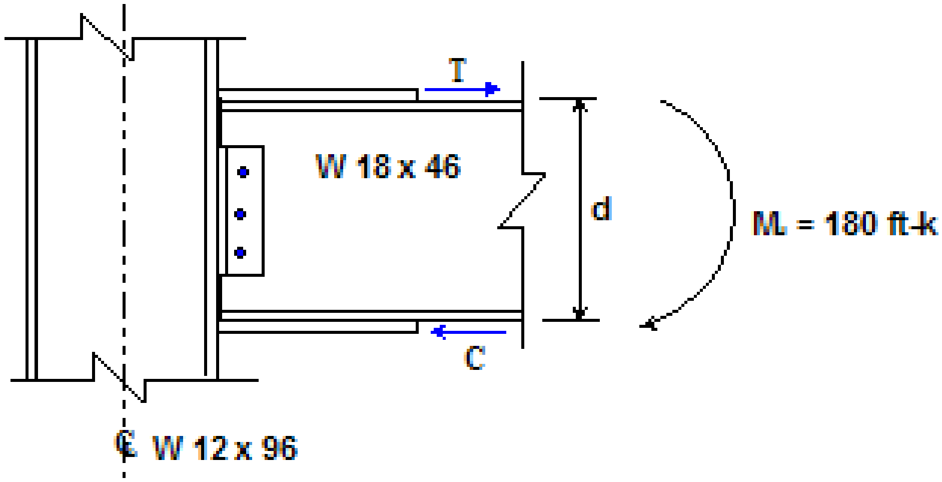


Figure 14

Solution:

Step 1 : Check the beam design flexural strength

Z_{req} = 180 x 12 / 0.9 x 50 = 48 in^3

Assuming two rows of 7/8 in. diameter A325-N bolts in standard holes, from LRFD Specification Section B10:

Beam W 18 x46: d = 18.1 in, b_f = 6.06 in, t_f = 0.605 in
Column W 12 x 96: d = 12.7 in, t_w = 0.550 in, t_f = 0.900 in)

$A_{fg} = b_f \times t_f = 6.06 \times 0.605 = 3.66\ in^2$

$A_{fn} = A_{fg} - 2\ (7/8 + 1/8)t_f = 3.66 - 2(1)0.605 = 2.45\ in^2$

Since $0.75F_uA_{fn}$ (119 kips) is less than $0.9F_yA_{fg}$ (164.7 kips), the effective tension flange area A_{fe} is:

$A_{fe} = (5/6)\ F_u/F_y\ \ A_{fn} = (5/6)(65/50)2.45 = 2.65\ in^2$

This is 27.6 % reduction from the gross flange area A_{fg}, and the effective plastic section modulus Z_e is approximated as:

$Z_e \sim Z_x\ - 2(0.276A_{fg}\ d/2) = 90.7 - 2(0.276 \times 3.66 \times 18.1\ /2)$
$= 72.4\ in^3 > Z_{req}\ (48\ in^3)$

The beam flexural design strength is OK

Step 2: Compute the magnitude of the internal compression and tension forces, T and C:

$C = T = 180 \text{ x}12 / 18.1 = 120$ kips

Step 3: Design the tension flange plate and connection

$P_{uf} = 120$ kips

For a bolted flanged-plated connection, determine the number of 7/8 in. diameter A325-N bolts required for shear:

From LRFD Table 7-10:

$\phi R_n = 21.6$ kips / bolt in single shear

number of bolts required, $n = 120 / 21.6 = 5.55$ ----→ 6 bolts

Try plate ¾ in. x 6 in. ASTM A36

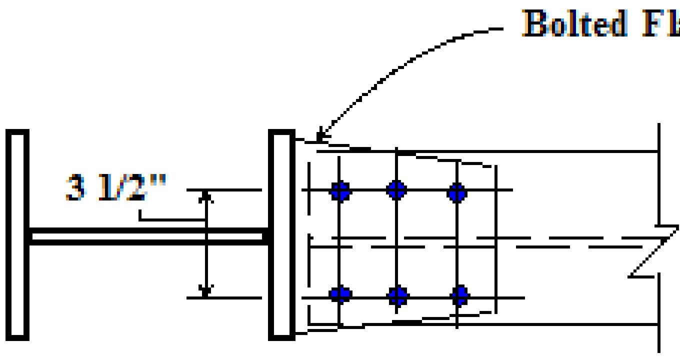
Find number of 7/8 in. diameter A325-N bolts required for material bearing on beam flange (more critical than flange plate), from LRDF Table 7-12:

The design bearing strength at bolt holes for standard size hole, material tensile strength $F_u = 65$ ksi and bolt spacing s = 3 in.

$\phi R_n = 102$ kips / in. thickness, and since $t_f = 0.605$ in, then

$\phi R_n = 102 \text{ x } 0.605 = 61.71$ kips / bolt

Therefore bolt shear is more critical. Try two rows of three bolts on a 3 ½ in. gage.



Check tension yielding of flange plate:

$\phi R_n = \phi F_y A_g = 0.9 \times 36 \times 6 \times 0.75 = 145.8$ kips > 120 kips

Check Tension rupture of flange plate:

$\phi R_n = \phi F_u A_n = 0.75 \times 58 \times [6 - 2 \times (7/8 + 1/8)]\ 0.75 = 130.5$ kips > 120 kips

Check of block shear rupture of flange plate will be required; it is beyond the scope of this course (refer to Course No. 134 for more information)

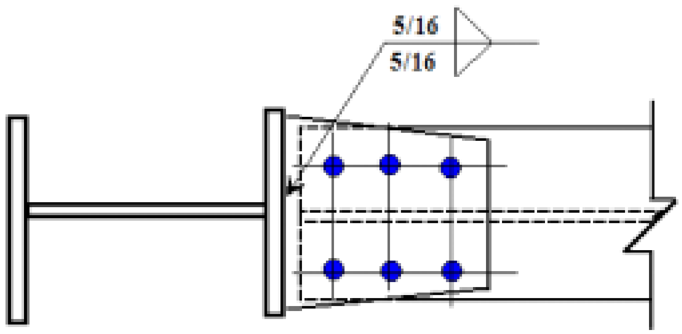
Compute the required weld size to supporting column flange:
The minimum fillet weld size required in multiple of 1/16th is given by:

$D_{min} = P_{uf} / 1.392 \times L_{weld}$ for E70 weld electrode

Welding both sides of the flange plate to the column flange the following is obtained:

$D_{min} = 120 / 2 \times 1.5 \times 1.392 \times 6 = 4.78$ -→ 5 sixteenths fillet weld is req'd

Note that the 1.5 factor is from LRFD Specification Appendix J2.4 for fillet weld loaded perpendicular to the weld longitudinal axis.



Step 4: Design the compression flange plate and connection

Compute the compressive strength of the flange plate by considering K = 0.65 and L = 2 in (1 ½ in. edge distance plus ½ in. setback)

The r for a ¾ in plate is given by

$$r = \sqrt{t/12} = \sqrt{\frac{0.75}{12}} = 0.25 \text{ in.}$$

Then, KL/r = 0.65 x 2 / 0.25 = 6

From LRFD Specification Table 3-36 (pg. 16.1-143)

ϕF_{cr} = 30.54 ksi for KL/r = 6

The design compressive strength of the flange plate is computed as

ϕR_n = 30.54 x 6 x 0.75 = 137.4 kips > 120 kips

The compression flange will be connected in the same manner as the tension flange with 6 – 7/8 in. diameter bolts in two rows of three bolts on a 3 ½ in. gage and 5/16 in. fillet welds to the supporting column flange.

The column must be check for stiffening requirements as described in Section 5.

Example 3

Design the moment-resisting connection of Example 2 using welded flanged plated FR moment connection.

Solution:
All steps are the same except that the flange plates will be welded to the flange of the supported beam (W18 x 46).

Determine the tension flange plate and connection:

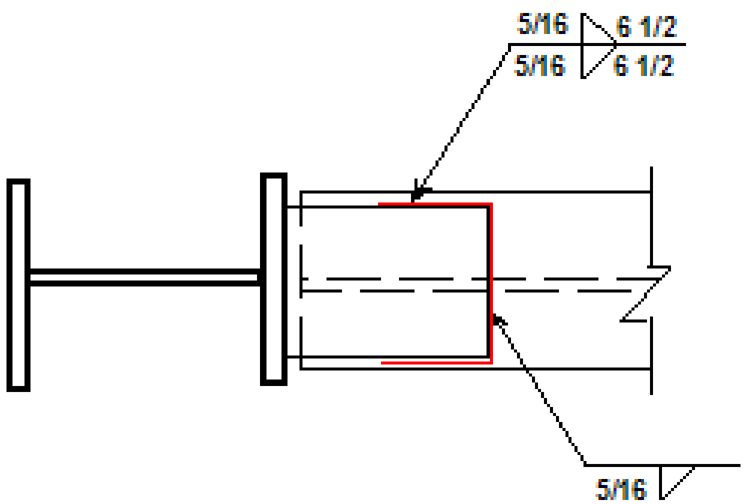
Assume a shelf dimension of 1/2 in. on both sides of the plate. The plate width is computed as b = 6.06 – 2(0.5) = 5.05 in.

Try a 3/4 in. x 5 in plate
Check tension yielding of flange plate:

$\phi R_n = \phi F_y A_g = 0.9 \times 36 \times 5 \times 0.75 = 121.5$ kips > 120 kips

Determine the required weld size and length of fillet welds to beam flange:
Try a 5/16 in. fillet weld, the minimum length of weld L_w required is:

$L_{min} = P_{uf} / 1.392 \times D = 120 / 1.392 \times 5 = 17.24$ in



Use 6 ½ in. of weld along each side and 5 in. of weld along the end of the flange plate.
All other steps are similar are those shown in Example 2.

7. Directly Welded Flange FR Moment Connections

A directly welded flange FR moment connection consists of a shear connection and complete-joint penetration groove welds connecting the top and bottom flanges of the beam to the supporting column, see Figure No. 15.

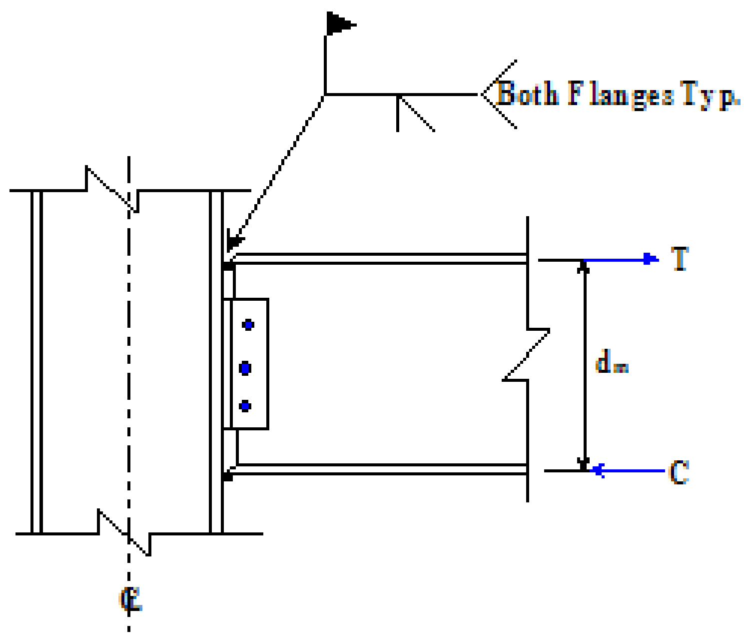


Figure 15

This moment connection is quite popular with fabricator, and its design is straightforward. The design consists in mainly computing the magnitude of the internal compression and tension forces T and C. These forces are assumed to be concentrated at the center of each beam flange as shown on Figure No. 15.

Next the areas of the full-penetration welds to the column are determined. The area is found as:

Area req'd = C or T / ϕF_y
Where ϕF_y is given in LRFD Specification Table J2.5, and for complete-joint penetration with tension and compression stresses normal to the weld effective area, $\phi = 0.9$ and F_y is the base material yield strength.

One of the most important considerations for this type of connection is weld shrinkage. This can cause erection problems in locating and plumbing the columns along lines of continuous beams. The typical complete-joint-penetration in a directly welded flange connection for a rolled beam may shrink about 1 /16 in. in the length of the beam when it cools and contracts. In addition, field welding should be arranged for welding in the flat or horizontal position. For more detailed information refer to AWS D1.1.

Example 4

Design a directly welded flange FR moment connection for the W24 x 84 beam shown in Figure No. 14. The beam specification is ASTM A992 grade 50, and the factored end moment is 590 ft-kips. Use E70 electrodes.

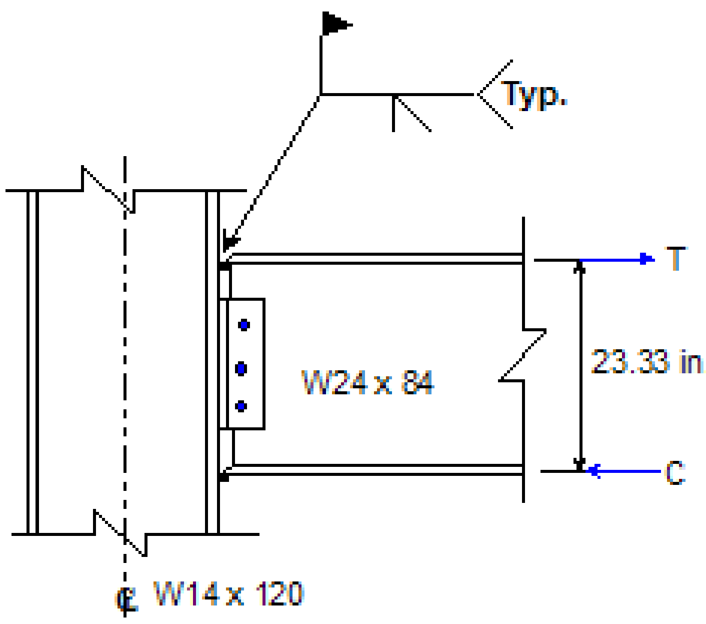


Figure 14

Beam W24 x 84: d = 24.1 in, b_f = 9.02 in, t_f = 0.770 in
Column W 14 x 120: d = 14.5 in, t_w = 0.590 in, t_f = 0.940 in)

C = T = 590 x 12 / 23.33 = 304 kips

Area of groove weld required = 304 / 0.9 x 50 = 6.76 in^2

Width required = 6.76 / 0.77 = 8.78 < 9.02 in.

USE 9 in. wide full penetration groove welds, E70.

The end shear connection needs to be design for the shear load at the connection and the column must be checked for stiffening requirements per LRFD Specification Chapter K as detailed in Section 5 of this course.