8 Design of Connections

8.1 RIVETED CONNECTIONS

Nowadays, riveted connections are rarely used. A designer may encounter riveted connections when analyzing an existing riveted structure for increased loading, for example. A single rivet is show in Fig. 8.1. It consists of a cylindrical shank and a rounded head. The other

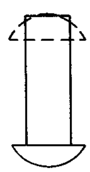


Figure 8.1

rounded head is usually formed by heating the rivet and using a rivet gun operated with compressed air.

According to ASTM (American Society for Testing and Materials) classifications, three types of rivets may be used in structural steel design (ASD TABLE J3.2).

A502, Grade 1. These rivets have a low carbon content and are more ductile than the ordinary structural steel and thus easier to drive.

Table 8.1 ALLOWABLE TENSILE AND SHEAR STRESSES FOR RIVETS (ASD TABLE J3.2)

Type of rivet	Allowable tensile stress (F_t)	Allowable shear stress (F_v) bearing-type connection
A502, Grade 1 hot-driven rivets	23 ksi	17.5 ksi
A502, Grade 2 and 3 hot-driven rivets	29 ksi	22 ksi

A502, Grade 2. They are carbon-manganese rivets and stronger than Grade 1.

A502, Grade 3. These rivets have the same strength as the Grade 2 but have better corrosion resistance.

Allowable tensile and shear stresses for rivets are given in Table 8.1. Rivets may not be used as friction-type or slip-critical connections, because the amount of friction between the rivet and the parts being connected is not dependable.

The allowable bearing stress for rivets is the same as for bolts and equal to (ASD J3.7) (when deformation around the hole is not a design consideration)

$$F_p = 1.5F_u \tag{8.1}$$

where F_u is the specified minimum tensile strength of the steel used in the connected parts. To simplify the design calculation, the bearing stress is assumed uniform over the projected area of bolts and rivets, that is, a rectangular area equal to the diameter of the rivet or bolt times the thickness of the plate.

8.2 BOLTED CONNECTION

Today, bolting has practically superseded riveting. Bolting is fast and does not require as much skilled labor as riveting or welding does.

8.2.1 High-Strength Bolts

Two types of high-strength bolts are commonly used for bolted connections in steel structures. They are designated as A325 and A490. These bolts have hexagon heads and use hexagon nuts. The common sizes for these bolts are in the range $\frac{1}{2}$ to $1\frac{1}{2}$ in. In steel buildings, the most common sizes appear to be $\frac{3}{4}$ in. and $\frac{7}{8}$ in.

Table 8.2 ALLOWABLE STRESSES FOR BOLTS (ASD TABLE J3.2)

Designation	Allowable tensile stress (F_t), ksi	Allowable shear stress (F_v), ksi	
		Slip critical (standard hole)	Bearing type
A307	20	-	10.0
A325N	44	17	21.0
A325X	44	17	30.0
A490N	54	21	28.0
A490X	54	21	40.0

The allowable tensile and shear stresses are summarized in Table 8.2. The letter N in the high-strength bolt designation indicates that threads are included in the shear plane, while the letter X indicates that threads are excluded from the shear plane.

The ASD provides two types of connections for high-strength bolts: slip critical or friction type and bearing type. In the slip critical connection sufficient slip resistance is provided under service conditions. In other words, a high factor of safety is used to prevent bearing of the bolt shank against the side of the hole. The bearing-type connection is used whenever occurrence of slippage under occasional overloads is tolerated. Evidently, allowable shear stresses for bearing-type connections are larger than the corresponding values for slip critical connections. In Table 8.2, the allowable shear stress values for the slip critical high-strength bolts are for standard-size holes. For oversized, short-slotted, and long-slotted holes, the allowable shear stress is decreased as given in ASD Table J3.2.

The allowable bearing stress for bolts is the same as for rivets, as given by Eq. (8.1). It should be noted that experimental research on bolted connections has shown that neither the bolts nor the connected parts fail in bearing. The magnitude of the bearing stress has an influence on the efficiency of the connection, however. Thus, the ASD code requirement for checking bearing stresses is to take into account the decreased efficiency of the connection due to high bearing stresses.

Due to the larger allowable shear stress, the bearing-type connection normally yields the more economical solution. The friction-type connection is used in structures subjected to impact and vibration resulting in considerable stress variations of reversals.

8.2.2 Unfinished Bolts

Unfinished bolts, also called common bolts or rough bolts, are made of low-carbon steel and designated by A307. They are cheaper than high-strength bolts and can be used for static loading only. They are usually used in light structures and secondary or bracing members. They usually have square heads and nuts to reduce the cost. A connection made of unfinished bolts may not necessarily be less expensive than an equivalent connection made of high-strength bolts, because the required number of bolts is usually larger for unfinished bolts. These bolts are used in sizes varying from $\frac{1}{4}$ to 4 in. The allowable tensile and shear stresses for A307 bolts are given in Table 8.2 (ASD Table J3.2).

8.2.3 Bolts Subjected to Eccentric Shear

Consider the group of bolts shown in Fig. 8.2(a) subjected to a force F having an eccentricity of e from the centroid C of the bolt group. To find the magnitude of shear force in each bolt, it is convenient to decompose the loading in Fig 8.2(a) to the statically equivalent loading in Fig. 8.2(b), which in turn can be considered as the sum of the force F passing through the centroid C and a couple of magnitude Fe as shown in Fig. 8.2(c). The shear force in each bolt due to force F passing through the centroid is equal to F divided by the number of bolts N. Now we calculate the shear forces in the bolts due to the couple of magnitude Fe, using the following assumption:

1. The gusset plate is perfectly rigid.
2. The bolts are elastic.

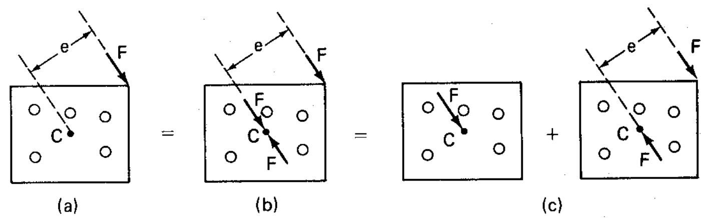


Figure 8.2

The couple $M_c = Fe$ causes the plate to rotate about the centroid of the bolt group. This rotation produces strains in the bolts which are proportional to their distances from the centroid. Since bolts are assumed to behave elastically, shear forces $R_1, R_2, \ldots, R_5$ developed in the bolts are also proportional to their corresponding distance $r_1, r_2, \ldots, r_5$ from the centroid (Fig. 8.3). In other words, if we denote the small angle of rotation of the plate by θ (in radian) and displacement of bolt j in the direction of shear force R_j by d_j, we can write

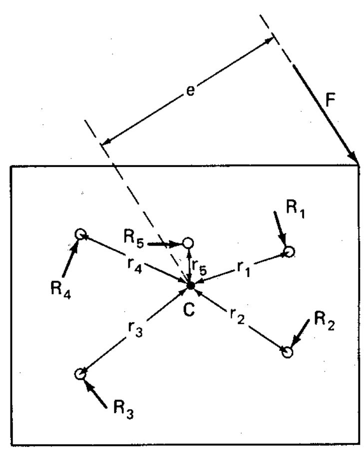


Figure 8.3

$$d_j = r_j\theta \tag{8.2}$$

$$R_j = Kd_j = K\theta\, r_j \tag{8.3}$$

$$\frac{R_1}{r_1} = \frac{R_2}{r_2} = \cdots = \frac{R_j}{r_j} = K\theta \tag{8.4}$$

In Eqs. (8.3) and (8.4), K is a proportionality factor. By writing the moment equilibrium we obtain

$$M_c = \sum_{i=1}^{N} R_i r_i = K\theta \sum_{i=1}^{N} r_i^2 \tag{8.5}$$

Deleting $K\theta$ between Eqs. (8.3) and (8.5), we will have.

$$R_j = \frac{M_c r_j}{\sum_{i=1}^{N} r_i^2} \tag{8.6}$$

This shear force R_j is normal to the line drawn from the centroid C to the bolt j. It is often easier to use the vertical and horizontal components of the shear R_j. By selecting the origin of coordinates at the centroid C (Fig. 8.4) we can write

$$R_{jx} = R_j \sin\alpha = R_j \frac{-y_j}{r_j} = -\frac{M_c y_j}{\sum_{i=1}^{N} r_i^2}$$

$$R_{jy} = R_j \cos\alpha = R_j \frac{x_j}{r_j} = \frac{M_c x_j}{\sum_{i=1}^{N} r_i^2}$$

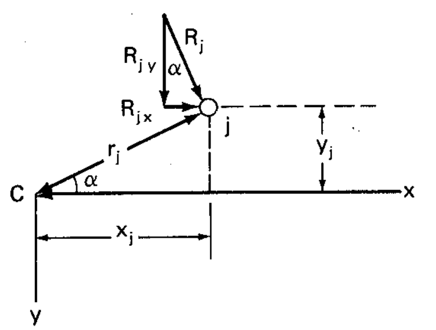


Figure 8.4

Finally, taking into account the contribution of the concentric force, we have

$$R_{jx} = \frac{F_x}{N} - \frac{M_c y_j}{\sum_{i=1}^{N} r_i^2} \tag{8.7}$$

$$R_{jy} = \frac{F_y}{N} + \frac{M_c x_j}{\sum_{i=1}^{N} r_i^2} \tag{8.8}$$

where F_x and F_y are the x and y components of the force F, respectively. Note that in deriving Eqs. (8.7) and (8.8), it is assumed that the positive x-axis is to right and the positive y-axis is downward.

8.2.4 Bolts Subjected to Combined Shear and Tension

Experiments on bolts subjected to combined shear and tension in bearing type connections have demonstrated that their strength can be

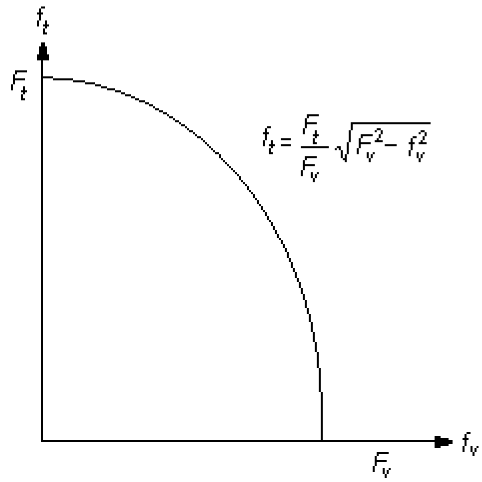


Figure 8.5

represented by an elliptical interaction curve with the major axis half-length equal to the allowable stress F_t and the minor axis half-length equal to the allowable stress F_v given in Table 8.2 (ASD Commentary J3.5) as shown if Fig. 8.5. Thus, according to ASD J3.5 the allowable tensile stress F_t for bolts subjected to combined shear and tension in bearing type connections is given in terms of the actual stress f_v in Table 8.3.

TABLE 8.3 ALLOWABLE TENSILE STRESS FOR BOLTS IN COMBINED SHEAR AND TENSION IN BEARING TYPE CONNECTION

Type	F_t
A325N	$\sqrt{44^2 - 4.39 f_v^2}$
A325X	$\sqrt{44^2 - 2.15 f_v^2}$
A490N	$\sqrt{54^2 - 3.75 f_v^2}$
A490X	$\sqrt{54^2 - 1.82 f_v^2}$

TABLE 8.4 MINIMUM BOLT PRETENSIONS (equal to 70 percent of the minimum tensile strength of bolts rounded off to nearest Kip) (ASD J3.6)

Bolt size (in.)	A325 Bolts	A490 Bolts
1/2	12	15
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
$1\frac{1}{8}$	56	80
$1\frac{1}{4}$	71	102
$1\frac{3}{8}$	85	121
$1\frac{1}{2}$	103	148

When a slip-critical bolted connection is subjected to tension in addition to shear, the clamping force is reduced and therefore the allowable shear stress F_v must also be reduced to take into account the loss of pretension. The reduction factor given in ASD J3.6 is $1\text{-}f_t A_b / T_b$ and the allowable shear stress in the slip critical connection is given by

$$F_v = \left(1 - \frac{f_t A_b}{T_b}\right)(17) \qquad \text{for A325 bolts} \qquad (8.9)$$

$$F_v = \left(1 - \frac{f_t A_b}{T_b}\right)(21) \qquad \text{for A490 bolts} \qquad (8.10)$$

where f_t is the actual tensile stress in the bolt, A_b is the cross-sectional area of the bolt, and T_b is the specified pretension load of the bolt given in Table 8.4.

In the case of combined dead and live loads and wind (or seismic) load, the constants in Table 8.3 shall be increased 33 percent, but the coefficients applied to f_v shall be kept the same. The reduced allowable shear stress given by Eq. (8.9) or (8.10) shall also be increased 33 percent.

8.2.5 Examples

Example 1

For the connection shown in Fig 8.6, find the number of $\frac{7}{8}$ in. A490 bearing-type bolts, assuming that threads are excluded from the shear planes. Plates are made of A36 steel with yield stress of 36 ksi and ultimate stress of F_u = 58 ksi.

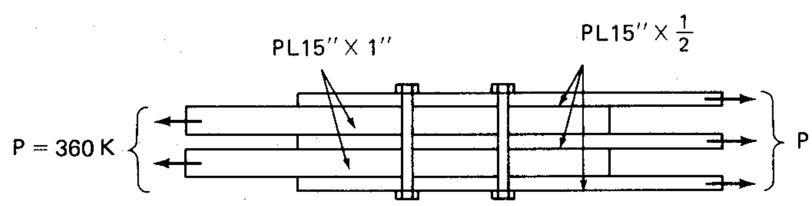


Figure 8.6

<u>Solution</u>

The free-body diagram for one bolt is shown is Fig. 8.7. Denoting the number of bolts by *N*, maximum shear in the bolts is *P/3N*. The allowable shear stress for the bolts is F_v = 40 ksi (Table 8.2). For finding the cross-sectional area of one bolt A_b, the table of square and round bars in the ASD manual may be used.

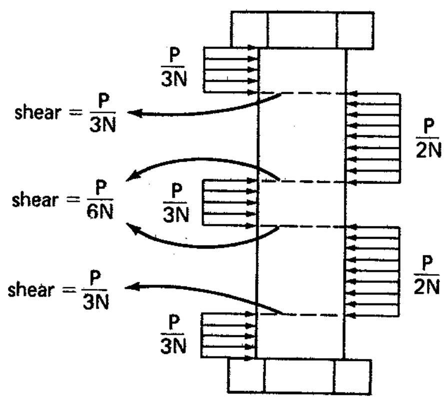


Figure 8.7

$$A_b = 0.60\,\text{in.}^2$$

$$A_b F_v = (0.60)(40) = 24\,\text{Kips} = \frac{P}{3N} = \frac{360}{3N} \Rightarrow N = 5$$

The allowable bearing stress from Eq. (8.1) is (assuming deformation around the hole is not a design consideration)

$$F_p = 1.5F_u = 1.5(58) = 87\,\text{ksi}$$

The bearing stress on the plate 15 in. x $\frac{1}{2}$ in. is larger than that on the plate 15 in. x 1in. Hence, check bearing on PL 15 in. x $\frac{1}{2}$ in.

$$f_p = \frac{P/3N}{dt} = \frac{P}{3Ndt} = \frac{360}{3(5)(\frac{7}{8})(\frac{1}{2})} = 54.9 \text{ ksi} < F_p \qquad \underline{\textbf{O.K.}}$$

Therefore, use N = 5.

Example 2

A built-up girder is made of a W21x93 section and two C12x30 sections, as shown in Fig. 8.8. In each plane of connection, the three sections are connected by four bearing-type A490 high-strength bolts of 0.5-in. diameter. Determine the pitch (longitudinal spacing of the bolts) where the shear force on the girder is *V* = 100 Kips. Assume that bolt threads are excluded from the shear planes.

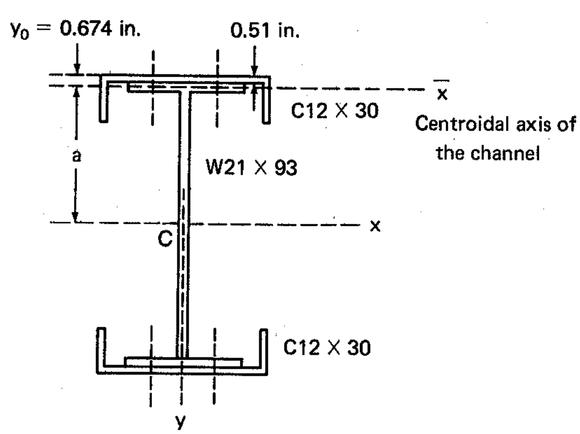


Figure 8.8

Solution

Properties of W21x93: d = 21.62 in.

I_x = 2070 in.^{4} t_f = 0.93 in. b_f = 8.42 in.

Properties of C12x30: A = 8.82 in.^{2}

y_o = 0.674 in. I_x = 5.14 in.^{4} t_w = 0.51 in.

The bolts must carry the longitudinal shear on the plane between the channel and the W shape. The total shear force between the channel and the flange of the W shape for a unit length of the beam is equal to

$$q = \frac{VQ}{I}$$

where Q is the first moment of the area of the channel about the centroidal axis of the built-up section and I is the moment of inertia of the built up section.

$$a = \frac{21.62}{2} + 0.51 - 0.674 = 10.646 \text{ in.}$$

$$I = 2070 + 2[5.14 + 8.82(10.646)^2] = 4079.55 \text{ in.}^4$$

$$Q = Aa = (8.82)(10.646) = 93.90 \text{ in.}^3$$

$$q = \frac{VQ}{I} = \frac{(93.90)(100)}{4079.55} = 2.30 \text{ Kips/in.}$$

Allowable shear stress for bearing-type high-strength A490 bolts is (Table 8.2)

$$F_v = 40 \text{ ksi}$$

Cross-sectional area of one bolt is

$$A_b = \frac{\pi d^2}{4} = \frac{\pi (0.5)^2}{4} = 0.1963 \text{ in.}^2$$

Denoting the longitudinal spacing of the bolts by p, we find that the total horizontal shear between the channel and the flange of the W shape over a length of p is qp. The shear capacity of the bolts over the same length is $2A_bF_v$. Therefore,

$$2A_bF_v = qp$$

$$p = \frac{2A_bF_v}{q} = \frac{2(0.1963)(40)}{2.30} = 6.83 \text{ in.}$$

According to ASD D2 and E4, the maximum longitudinal spacing of bolts connecting two rolled shapes in contact with each other shall not be greater than 24 in. Therefore, use $p = 6\frac{3}{4}$ in.

Example 3

Using A490 bearing-type connection, find the bolt size for the connection shown in Fig. 8.9. The column is a W14x99, and the thickness of the gusset plate is one inch. Both column and gusset plate are made of A36 steel with yield stress of 36 ksi. Bolt threads are excluded from the shear plane.

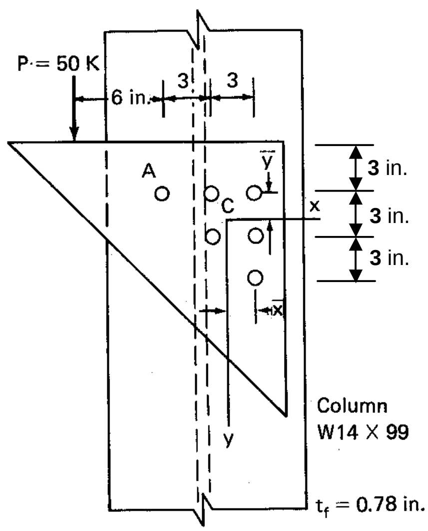


Figure 8.9

<u>Solution</u>

First, we must find the centroid of the bolt group (point C in Fig 8.9). Denoting the cross-sectional area of one bolt by A_b, the location of the centroid is determined by finding the distances $\bar{x}$ and $\bar{y}$ and shown in Fig. 8.9.

$$\bar{x} = \frac{6A_b + 2(3)A_b}{6A_b} = 2\,\text{in.}$$

$$\bar{y} = 2\,\text{in.}$$

The eccentricity of the load from the centroid is

$$e = -(6+3+3-2) = -10\ \text{in.}$$

and the magnitude of the corresponding couple is

$$M = Pe = -(50)(10) = -500\ \text{K - in.}$$

The horizontal and vertical components of the shear force R_j acting on the bolt j can be calculated from Eqs. (8.7) and (8.8).

$$P_x = 0 \qquad P_y = 50\ \text{K} \qquad N = 6$$

$$\sum_{i=1}^{N} r_i^2 = 2(4^2+2^2)+(1^2+1^2)+2(2^2+1^2)+(2^2+2^2) = 60\ \text{in.}^2$$

$$R_{jx} = \frac{F_x}{N} - \frac{M_c y_j}{\sum_{i=1}^{N} r_i^2} = \frac{500}{60} y_j = \frac{25}{3} y_j$$

$$R_{jy} = \frac{F_y}{N} + \frac{M_c x_j}{\sum_{i=1}^{N} r_i^2} = \frac{50}{6} - \frac{500}{60} x_j = \frac{25}{3}(1 - x_j)$$

The most stressed bolt is bolt A, for which x_A = -4 in. and y_A = -2 in. Therefore, the resultant shear force acting on this bolt is

$$R_A = \tfrac{25}{3}\sqrt{y_A^2 + (1-x_A)^2} = \tfrac{25}{3}\sqrt{2^2+5^2} = 44.88\ \text{Kips}$$

The design of bolts is commonly based on the most stressed bolt. The allowable shear stress for A490X bearing-type connections is F_v = 40 ksi (Table 8.2). Thus, the required cross-sectional area for one bolt is

$$\text{Required } A_b = \frac{R_A}{F_v} = \frac{44.88}{40} = 1.12 \text{ in.}^2$$

Try 1 1/4 in. bolts, A_b = 1.23 in.^{2}. Noting that the bolts are in single shear and the thickness of the column flange (t_f = 0.78 in.) is less than the thickness of the gusset plate, we should check the bearing in the column flange for t = 0.78 in.

$$f_p = \frac{R_A}{dt} = \frac{44.88}{(1.25)(0.78)} = 46.03 \text{ ksi} < F_p = 1.5F_u = 1.5(58) = 87 \text{ ksi}$$

O.K.

USE 1¼-in. bolts

Note: Since the spacing of bolts is less than minimum, i.e. $(2^2/_3)d$ = $(2^2/_3)(1.25)$ = 3.33 in., we need to increase the bolt spacing to 3.5 in.

8.3 WELDED CONNECTIONS

8.3.1 Types of Welding

In welded connections, different elements are connected by heating their surfaces to a plastic or fluid state. Notwithstanding the availability of both gas and arc welding, welded connections in steel structures are ordinarily done by arc welding. To obtain satisfactory connections, additional metal is used for joining different elements. In electric arc welding, the additional material is a metallic rod, which is used as the electrode. In this type of welding, the electric arc produced between the elements being welded and the electrode heats the elements and the electrode to the melting point. This transformation of electrical energy into thermal energy and the resulting high temperature (up to 10,000 °F) causes the metallic electrode to melt off into the joint. Small droplets of the molten metallic electrode are in fact driven onward to the joint. Thus, overhead welding is possible by electric arc welding.

Molten steel must be protected from the surrounding air; otherwise, gases contained in the molten steel can combine chemically with oxygen and nitrogen in the air. This chemical reaction leaves small pockets of gases in the weld after it has cooled down, making it porous. The resulting weld will be brittle with very little resistance to corrosion. To prevent this undesirable brittleness of the weld, two types of arc welding are commonly used. One is called Shielded Metal Arc Welding (with acronym SMAW) and the other is Submerged (or hidden) Arc Welding (with acronym SAW).

In SMAW, the weld is protected by using an electrode covered with a layer of mineral compounds. Melting of this layer during the welding produces an inert gas encompassing the weld area. This inert gas

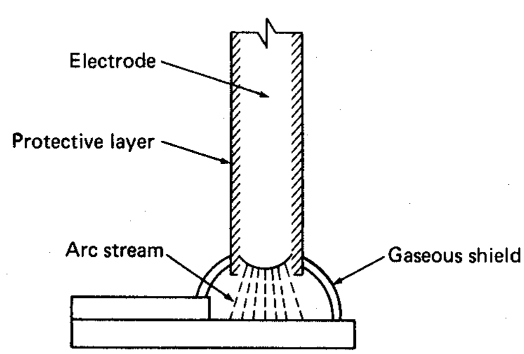


Figure 8.10 Shield metal arc welding (SMAW)

shields the weld by preventing the molten metal from having contact with the surrounding air (Fig. 8.10). The protecting layer of the electrode leaves a slag after the mold has cooled down. The slag can be removed by peening and brushing.

In SAW, the surface of the weld and the electric arc are covered by some granular fusible material and thus is protected from the surrounding air. In this method, a bare metal electrode is used as filler material. Compared with SMAW, SAW welds provide deeper penetration, and this fact is the allowable shear stress values recommended by ASD J2.2a. Also, SAW welds show good ductility and corrosion resistance and high impact strength.

8.3.2 Advantage of Welding

1. In welded connections, in general, fewer pieces are used. This will speed up the detailing and fabrication process.
2. In welded connections, gusset and splice plates may be eliminated. Bolts or rivets are not needed either. Thus, the total weight of a welded steel structure is somewhat less than that of the corresponding bolted structure.
3. Connecting unusual members (such as pipes) is easier by welding than by bolting.
4. Welding provides truly rigid joint and continuous structures.

One possible drawback of welding is the need for careful execution and supervision. For this reason, welding is sometimes done in the shop and bolting in the field. In other words, shop-welding is complemented by the bolting in the field.

8.3.3 Types of Welds

The two common types of welds in welded steel structures are *groove welds* and *fillet welds*. Fillet welds are much more popular in structural steel design than groove welds. Two different types of groove

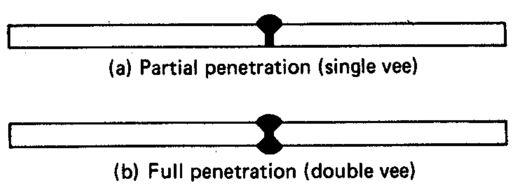


Figure 8.11 Groove welds. (a) Partial penetration (single vee); (b) full penetration (double vee)

welds are shown in Fig. 8.11. They are the partial penetration (single vee) groove weld and full penetration (double vee) groove weld. Groove welds can be used when the pieces to be connected can be lined up in the same plane with small tolerances.

Fillet welds are shown in Fig. 8.12. Depending on the direction of the applied load and the line of the fillet weld, fillet welds are classified as longitudinal or transverse fillet weld. In the former, the

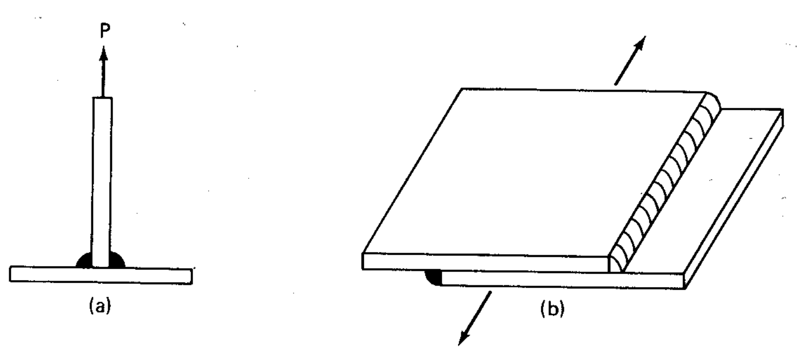


Figure 8.12 Fillet welds. (a) Transverse fillet; (b) longitudinal fillet

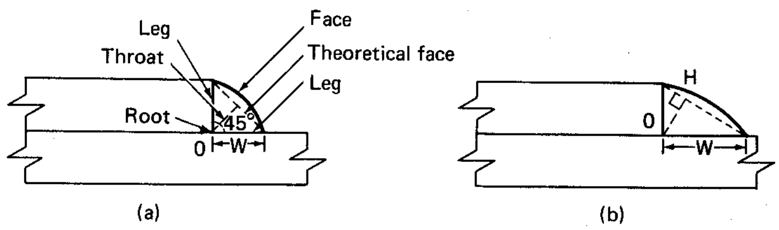


Figure 8.13 Fillet welds. (a) Equal leg; (b) unequal leg

shear force to be transferred is parallel to the weld line; in the latter, the force to be transmitted is perpendicular to the weld line.

Fillet welds can be either equal-leg or unequal-leg, as shown in Fig. 8.13. The intersection point of the original faces of the steel elements being connected is called the *root* of the weld. The surface of the weld should have a slight convexity. In computation of the strength of the weld, however, this convexity is not taken into account and the theoretical flat surface is used. A convex surface for a weld is clearly superior to a concave surface. When the weld cools down, it shrinks. This shrinkage causes surface tension in concave welds and surface compression in convex welds (Fig. 8.14). The concave surface in tension tends to crack, causing the separation of the weld from the faces of the pieces being connected. The normal distance from the root to the theoretical face of the weld is called the *throat* of the weld.

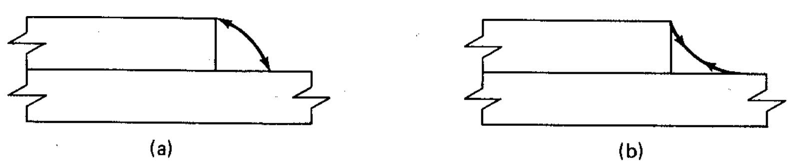


Figure 8.14 Fillet welds with convex and concave surface. (a) Convex surface under compression; (b) concave surface under tension

Experiments performed on fillet welds indicate that they are weaker in shear than in tension and compression. Also, equal-leg fillet welds fail in shear through the throat (at angles of about 45 degree with the legs of the weld). For equal-leg fillet welds, the relation between the dimensions of the leg *w* and the throat *t* is

$$t = 0.707\, w \tag{8.11}$$

Thus, shear stress is the controlling factor in the design of fillet welds; it is customarily calculated by dividing the force *P* acting on the weld by the effective throat area of the weld. The effective throat area is computed by multiplying the throat thickness by the length of the fillet weld. This method of finding average shear stress is used for both longitudinal and transverse fillet welds.

The ASD code does not recognize the fact that transverse fillet welds are stronger (about one-third) than the longitudinal fillet welds. Experiments indicate that transverse fillet welds fail in planes somewhat different from the 45-degree plane. The size of a fillet weld is indicated by the size of its leg. For example, a 7/8 in. fillet weld means a fillet weld with a leg size of *w* = 7/8 in.

As pointed out previously, SAW welds provide a deeper penetration than the SMAW welds. This fact is recognized in the ASD code by allowing a larger throat area for SAW welds. According to ASD J2.2a for SAW welds with size 3/8 in. or smaller, the effective throat thickness is taken as equal to leg size, and for fillet welds greater than 3/8 in., the effective throat is taken equal to the theoretical throat plus 0.11 in.

$$t = \begin{cases} w & \text{for } w \le 3/8 \text{ in.} \\ 0.707w + 0.11 & \text{for } w > 3/8 \text{ in.} \end{cases} \tag{8.12}$$

8.3.4 Allowable Stress on Welds

Allowable stresses for different types of welds are given in Table J2.5 of the ASD code. The allowable shear stress on the effective area of fillet welds is 0.30 times the nominal tensile strength of the weld metal. Electrodes are designated as E60, E70 and so on, where E stands for electrode and the number following the letter E is the minimum tensile strength of the weld, in ksi.

8.3.5 Minimum and Maximum sizes of Fillet Welds

The minimum size of fillet weld is determined on the basis of the thicker of the two pieces connected, as given in Table 8.5. (ASD J2.2b)

The maximum size of fillet welds along edges of an element less than ¼ in. thick is equal to the thickness of the element. Along edges of an elements with thickness of ¼ in. or more, the maximum size of the fillet weld is equal to the thickness of the element minus 1/16 in. (ASD J2.2b)

TABLE 8.5 MINIMUM SIZE OF FILLET WELDS (ASD TABLE J2.4)

Thickness of thicker part connected (in.)	Minimum leg size of fillet weld (in.)
To ¼ inclusive	1/8
Over ¼ to ½	3/16
Over ½ to ¾	¼
Over ¾	5/16

8.3.6 Minimum Length of Fillet Weld

The minimum effective length of a fillet weld shall be at least equal to four times its nominal size; otherwise weld size shall be limited to ¼ of its effective lengths (ASD J2.2b).

The effective length of any segment of intermittent fillet weld shall be at least 1.5 in. and four times the weld size (ASD J2.2b)

8.3.7 Eccentrically Loaded Welded Connections

The analysis of eccentrically loaded welded connections is similar to the analysis of eccentrically loaded bolts, as covered in Sec. 8.2.3. Consider a bracket welded to the flange of a column, as shown in Fig. 8.15. The size of the fillet weld is assumed to be the same on the three edges of the bracket. Suppose that the connection is subjected to horizontal and vertical shears of F_x and F_y that produce a moment M_c about the centroid C of weld lines. Thus, the total components of shear force per unit length at point A with coordinate x_A and y_A are

$$q_x = q_{xp} + q_{xm} = \frac{F_x}{L} - \frac{M_c y_A}{J} \tag{8.13}$$

$$q_y = q_{yp} + q_{ym} = \frac{F_y}{L} + \frac{M_c x_A}{J} \tag{8.14}$$

Finally, the resultant shear force per unit length of weld at point A is

$$q_A = \sqrt{q_x^2 + q_y^2} \tag{8.15}$$

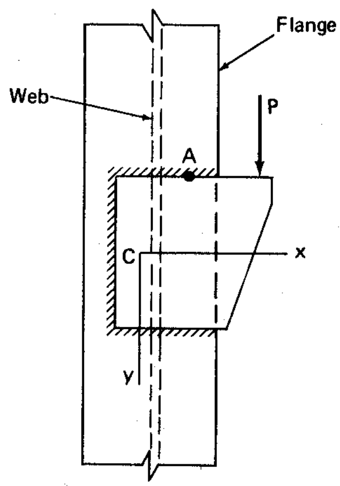


Figure 8.15

Note that in Eqs. (8.13) and (8.14) it is assumed that the positive x-axis points to the right and the positive y-axis points downward.

Denoting the total length of fillet weld by L, the horizontal shear force per unit length of the weld at any point A due to the direct shear F_x is

$$q_{xp} = \frac{F_x}{L}$$

and the vertical shear force per unit length of the weld due to the direct shear F_y is

$$q_{yp} = \frac{F_y}{L}$$

The shear force per unit length at point A (with coordinates x_A and y_A) due to couple M_c are

$$q_{xm} = -\frac{M_c y_A}{J}$$

$$q_{ym} = \frac{M_c x_A}{J}$$

where

$J = I_x + I_y$
= polar moment of inertia of the weld of unit width about point C
I_x = moment of inertia of the weld of unit width about the x-axis
I_y = moment of inertia of the weld of unit width about the y-axis

8.3.8 Examples

Example 4

A welded built-up girder is made of a W24x94 and a C12x25 section, as shown in Fig 8.16. The maximum shear force in the girder is V = 150 Kips. Design the intermittent fillet weld for connecting the two sections, as shown in Fig 8.16. Use submerged arc welding (SAW) and E70 electrodes.

Solution

Properties of W24 x 94

A = 27.7 in.^{2} $\quad$ b_f = 9.065 in. $\quad$ t_f = 0.875 in.

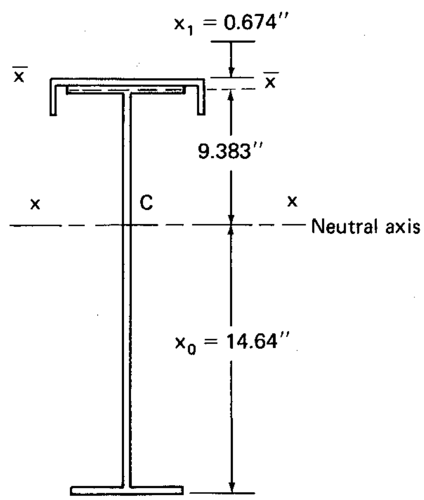


Figure 8.16

I_x = 2700 in.^{4} d = 24.31 in.

Properties of C12x25:

A = 7.35 in.^{2} d = 12.0 in. t_w = 0.387 in.

$I_{\bar{x}}$ = 4.47 in.^{4}

We first locate the neutral (centroidal) axis of the cross section of the built-up girder (x-axis in Fig 8.16).

$$x_0 = \frac{\sum yA}{\sum A} = \frac{(7.35)(24.31 + 0.387 - 0.674) + 27.7(24.31)/2}{7.35 + 27.7} = 14.64\,\text{in}$$

The moment of inertia of the built-up section about the x-axis is

$$I_x = 2700 + 27.7(14.64 - 24.31/2)^2 + 4.47 + 7.35(9.383)^2 = 3522.62\,\text{in}^4$$

The two lines of fillet weld must carry the longitudinal shear on the plane between the channel and the W shape. The total shear force between the channel and the flange of the W shape for a unit length of the beam is equal to

$$q = \frac{VQ}{I_x}$$

where Q is the first moment of the area of the channel about the centroidal axis of the built-up section.

$$Q = (7.35)(9.383) = 68.97 \text{ in.}^3$$

$$q = \frac{(150)(68.97)}{3522.62} = 2.94 \text{ K/in.}$$

- Minimum size of the fillet weld (Sec. 8.3.5): 5/16 in.

Use $\boxed{w = 5/16 \text{ in.}}$

- Maximum size of the fillet weld along the flange edge of the W shape (Sec. 8.3.5):

0.875 – 1/16 = 0.813 in.

- The minimum length of a segment of intermittent welds (Sec. 8.3.6): the larger of $4w = 1.25$ and 1.5 in.

Use $\boxed{L_1 = 1.5 \text{ in.}}$ Length of a segment of weld

- The effective throat thickness (Eq. (8.12)):

$$t = w = 5/16 \text{ in.}$$

- The allowable shear stress of the weld is

$$F_v = 0.30(70) = 21 \text{ ksi}$$

If the longitudinal spacing of intermittent welds is denoted by a, the total horizontal shear between the channel and the flange of W shape over a length of a is aq. The shear capacity of the two lines of fillet weld over the same length is $2L_1F_vw$. Therefore,

$$aq = 2L_1F_v w.$$

$$a = \frac{2L_1F_v w}{q} = \frac{2(1.5)(21)(5/16)}{2.94} = 6.7 \text{ in.}$$

- The maximum longitudinal spacing of intermittent welds connecting two rolled shapes in contact (ASD E4): a_{max} = 24 in.

Use $\boxed{a = 6.5 \text{ in.}}$

Example 5

Determine the size of the submerged arc fillet weld for the connection of a C12 x 25 beam to a W14 x 120 column as shown is Fig. 8.17. Note that the beam is connected to the column at its end and at the edge of the column flange. Use E70 electrodes.

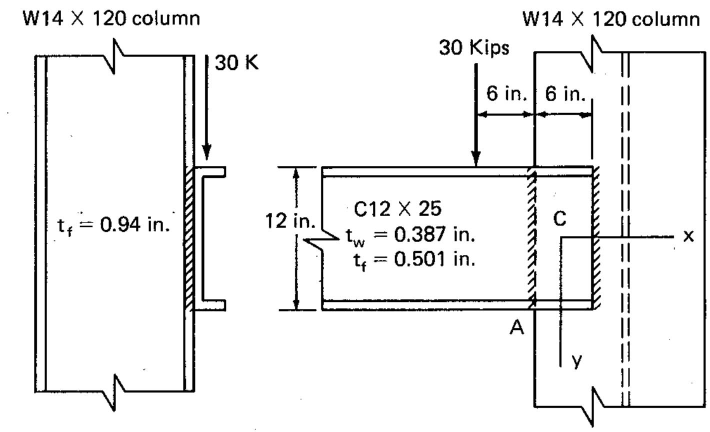


Figure 8.17

Solution

We design the weld on the basis of the maximum shear stress, which is at point A (Fig. 8.17).

x_A = -3 in. y_A = 6 in.

$F_x = 0$ F_y = 30 Kips M_C = -30(9) = -270 K-in.

Total length of the weld: L = 2(12) = 24 in.

$$I_x = 2(\tfrac{1}{12})(1)(12)^3 = 288 \text{ in.}^4/\text{in.}$$

$$I_y = 2(12)(3)^2 = 216 \text{ in.}^4/\text{in.}$$

$$J = I_x + I_y = 504 \text{ in.}^4/\text{in.}$$

$$q_x = \frac{F_x}{L} - \frac{M_c y_A}{J} = \frac{(270)(6)}{504} = 3.21 \text{ K/in.}$$

$$q_y = \frac{F_y}{L} + \frac{M_c x_A}{J} = \frac{30}{24} + \frac{(270)(3)}{504} = 2.86 \text{ K/in.}$$

$$q_A = \sqrt{q_x^2 + q_y^2} = \sqrt{3.21^2 + 2.86^2} = 4.30 \text{ K/in.}$$

For SAW and E70 electrode, the allowable shear stress is

$$F_v = 0.30(70) = 21 \text{ ksi}$$

The required effective thickness of the weld throat is

$$t = \frac{q_A}{F_v} = \frac{4.30}{21} = 0.205 \text{ in.} = \frac{3.3}{16} \text{ in.} < \frac{3}{8} \text{ in.}$$

Therefore,

$$w = t = 1/4 \text{ in.}$$

- The minimum size of the fillet weld for 0.94 in.-thick plate (column flange) (Sec. 8.3.5): 5/16 in.
- The maximum size of the fillet weld along the edge of the beam web (Sec. 8.3.5): 0.387 – 1/16 = 0.325 in.

Use $\boxed{\text{5/16 in.}}$ SAW fillet weld.

Example 6

To make a moment-resisting joint in a rigid frame, a W21x147 girder is directly welded to a W14x145 column as shown in Fig. 8.18. The beam, made of A36 steel with yield stress of 36 ksi, has an end reaction of 194 Kips and an end bending moment of 210 K-ft. Using E70 electrodes, find the size of the submerged arc fillet weld (SAW) for the connection of beam flanges and web to the column flange.

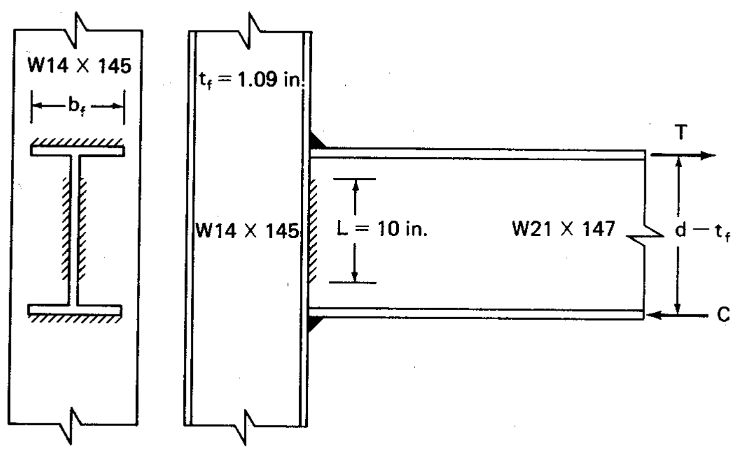


Figure 8.18

<u>Solution</u>

Assume that the web welds carry all of the shear and the flange welds carry all of the moment.

$$V = 194 \text{ Kips} \qquad M = 210 \text{ K-ft}$$

Properties of the beam:

d = 22.06 in. t_f = 1.15 in. t_w = 0.72 in. b_f = 12.51 in.

The allowable shear stress of the weld: F_v = 0.30(70) = 21 ksi.

(a) Web Welds

If the effective throat thickness is denoted by t, the shear stress in the weld is

$$f_v = \frac{V}{2tL} \le F_v$$

The required throat thickness:

$$t = \frac{V}{2LF_v} = \frac{194}{2(10)(21)} = 0.462\,\text{in.} > 3/8\,\text{in.}$$

For w > 3/8 in.:

$$t = 0.707w + 0.11 = 0.462\,\text{in..}$$

$$w = 0.5\,\text{in.}$$

Minimum weld size (Sec. 8.3.5): w_{min} = 5/16 in.

Use $\boxed{w = ½\text{ in.}}$

(b) Flange Welds

The force T to be carried by the weld is found by dividing the beam end bending moment by the center-to-center distance of flanges:

$$T = \frac{M}{d - t_f}$$

$$f_v = \frac{T}{tb_f} = \frac{M}{tb_f(d - t_f)} \leq F_v$$

$$t = \frac{M}{b_f(d - t_f)F_v} = \frac{(210)(12)}{(12.51)(22.06 - 1.15)(21)} = 0.459 \text{ in.} > 3/8 \text{ in.}$$

$$t = 0.707w + 0.11 = 0.459 \text{ in.}$$

$$w = 0.49 \text{ in.} > w_{\min} = 5/16 \text{ in.}$$

Use $\boxed{w = ½ \text{ in.}}$

8.4 WEB-BASED INTERACTIVE DESIGN OF CONNECTIONS

8.4.1 Design of Simple Bolted Beam-Column and Beam-Girder Connections

Fig. 8.19 shows the initial screen of the applet for design of simple bolted beam-to-column and beam-to-girder connections according to the AISC ASD specifications. This applet consists of three panels: *Input* panel, *Results* panel, and *Graphic View* panel.

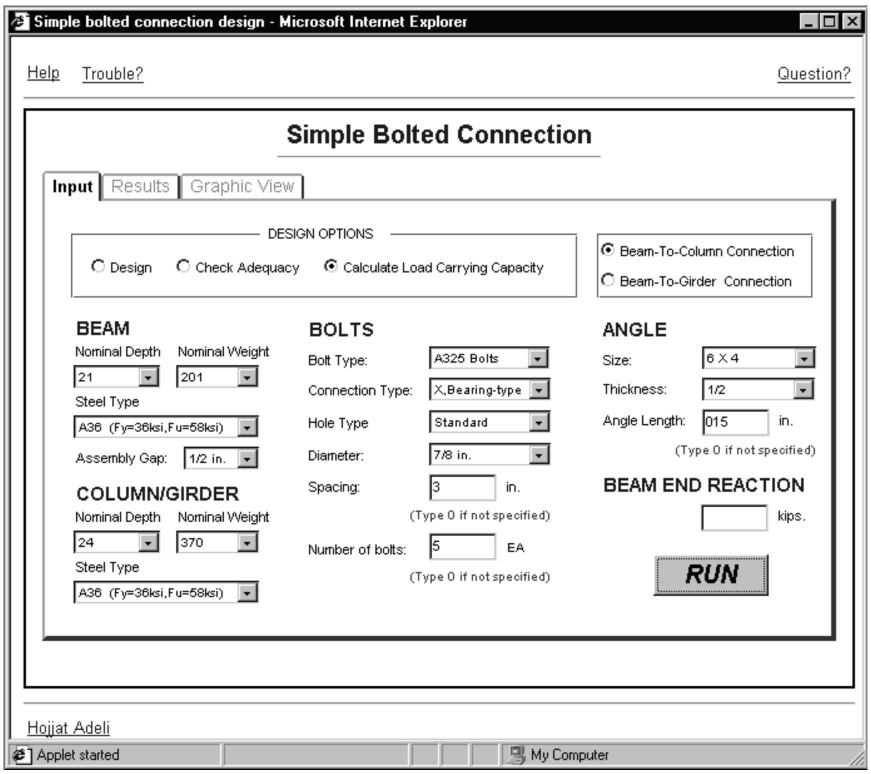
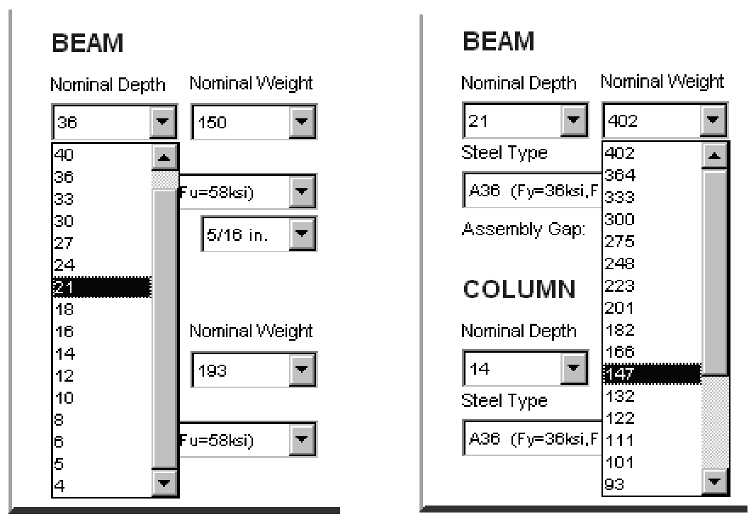


Figure 8.19 Input panel for the simple bolted connection design

The user can enter the connection design input in the input panel (Fig. 8.19). This input includes selection of the W (wide flange) shapes for beam and column, angle, bolt type (bearing type with threads excluded or included versus slip critical type), connection type (bearing type with threads excluded or included in the shear plane versus slip critical type), bolt hole type (standard, oversized, short-slotted, or long-slotted), bolt diameter, bolt spacing, number of bolts, and entering the end reaction. Two types of bolted connections are included: beam-to-column and beam-to-girder. The applet can design the connection, check



Step 1) Step 2)

Figure 8.20 Selection of a W shape; step 1) select the nominal depth, step 2) select the weight per unit length

the adequacy of a given connection for a specified beam end reaction, or calculate the shear load-carrying capacity of a given connection.

The user is allowed to choose among the 295 W shapes available in the AISC manuals (AISC, 1995 & 1998) for the beam and column. A

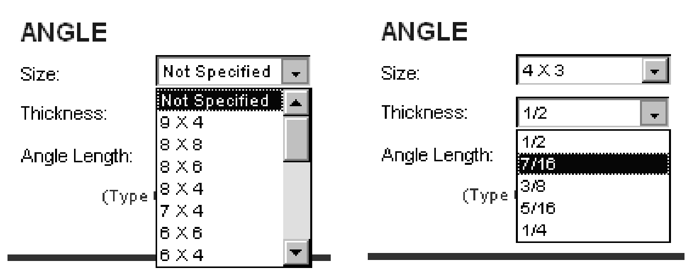


Figure 8.21 Selection of an angle

two-step input scheme is used to minimize the input entry by the user (Fig. 8.20). In step 1 the user is asked to select a nominal depth. In step 2 the user is asked to select the weight per unit length among the members with the selected nominal depth. The user also selects the type of the steel and the connection assembly gap (the clear distance between the beam and the column).

In the input panel the user selects the angle size, thickness, and length, the connection type, the bolt type, the hole type, and the diameter and the spacing of the bolts. Fig. 8.19 shows the default values for the

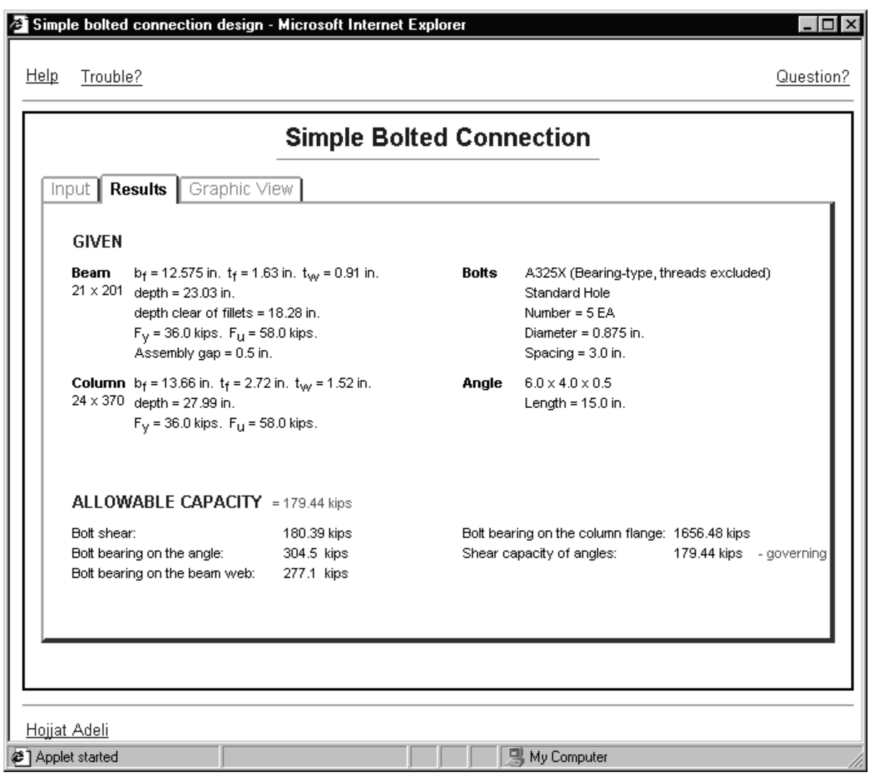


Figure 8.22 Results panel for the simple bolted beam-to-column connection design

angles and the bolts. The user can skip choosing any one of these sizes. In that case, the applet designs the lightest angle and the smallest bolt sizes using the commonly used increment of 1/8 in. A two-step scheme is also used for selection of the angle as shown in Fig. 8.21. Fig. 8.22 shows the results panel for finding the load carrying capacity of a simple beam-to-column bolted connection (for the example data given in Fig. 8.19). Fig. 8.23 shows the graphic view panel for this connection.

Figures 8.24, 8.25, and 8.26 show, respectively, the input, results, and graphic view panel for a simple bolted beam-to-girder

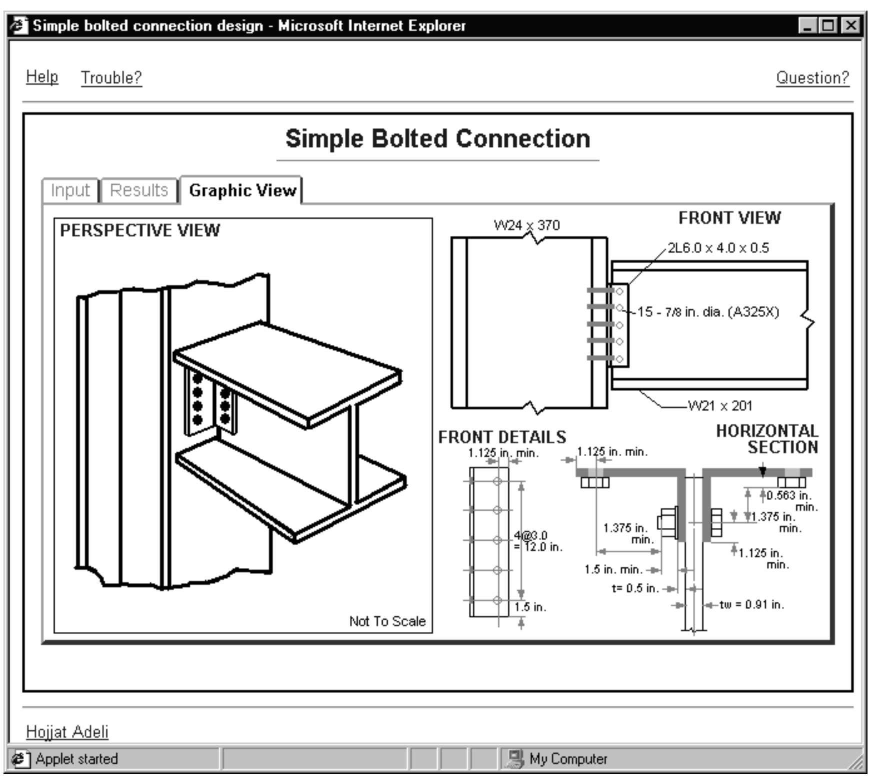


Figure 8.23 Graphic view panel for the simple bolted beam-to-column connection design

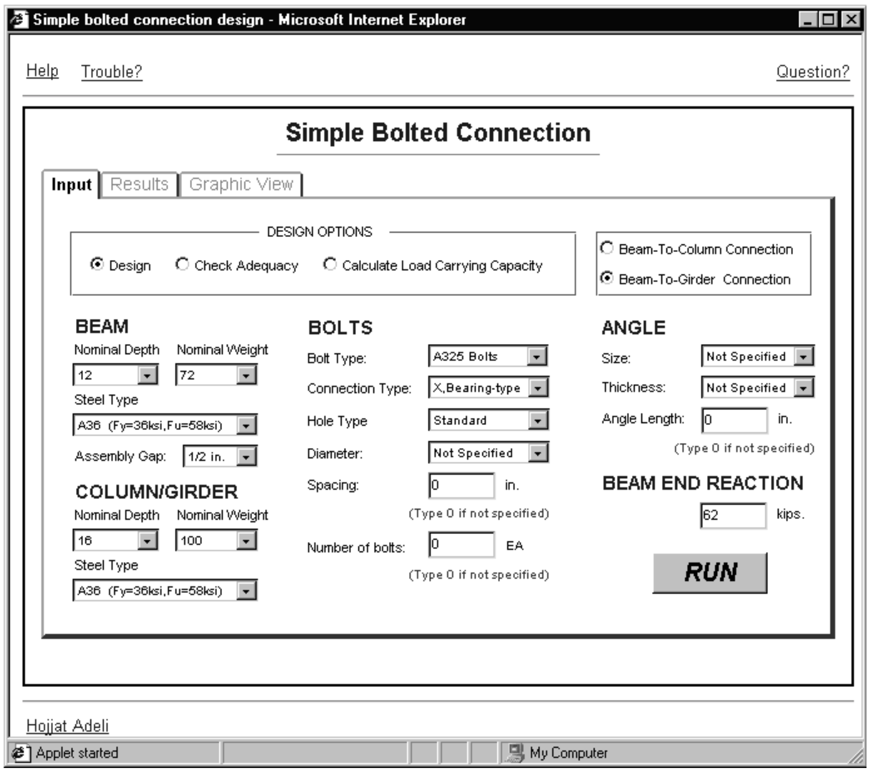


Figure 8.24 Input panel for the simple bolted beam-to-girder connection design

connection. The results and graphics view of Figs. 8.25 and 8.26 are found based on the example data in Fig. 8.24. In this case, the applet designs the beam-to-girder connection with the least amount of data entered in the input panel.

These figures display all the necessary details and useful information. A perspective view of the connection and front view and details of the connection with all the dimensions are displayed in the graphic view panel. The front view and details are generated by the applet in real time after the connection design is completed. The

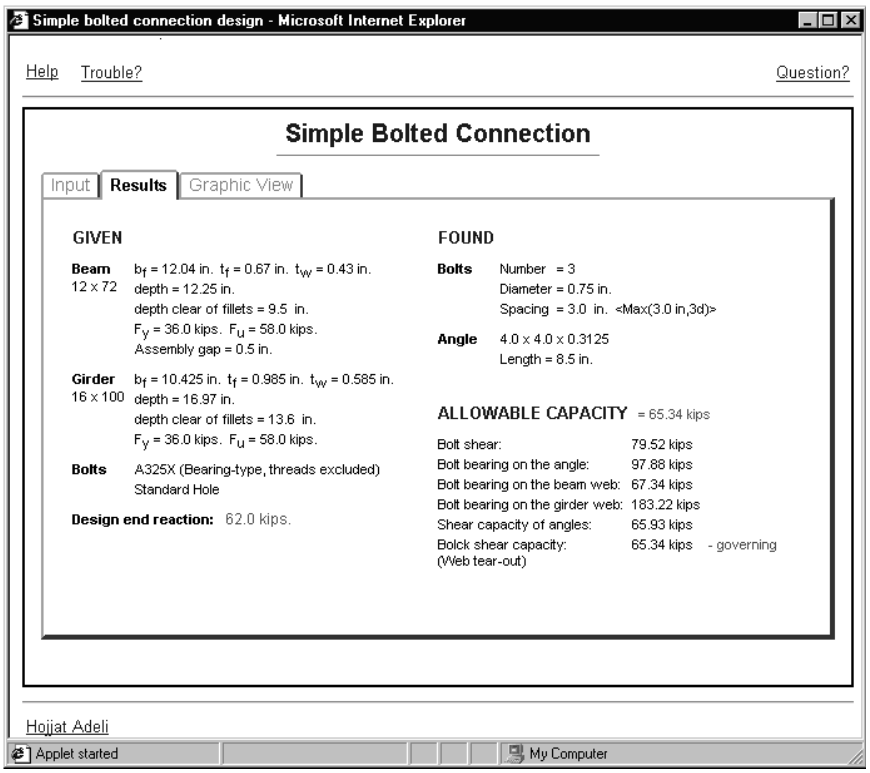


Figure 8.25 Results panel for the simple bolted beam-to-girder connection design

perspective view, on the other hand, is not drawn by the applet. It is a generic figure placed on the server where the applet source bite code resides.

Whenever the user clicks on the tab attached to the graphic view panel, the applet downloads the perspective of the connection from a database of connection perspectives stored on the server (Fig. 8.27). This approach saves time for real-time generation of a perspective view by the applet which involves a large amount of computations. To expedite the

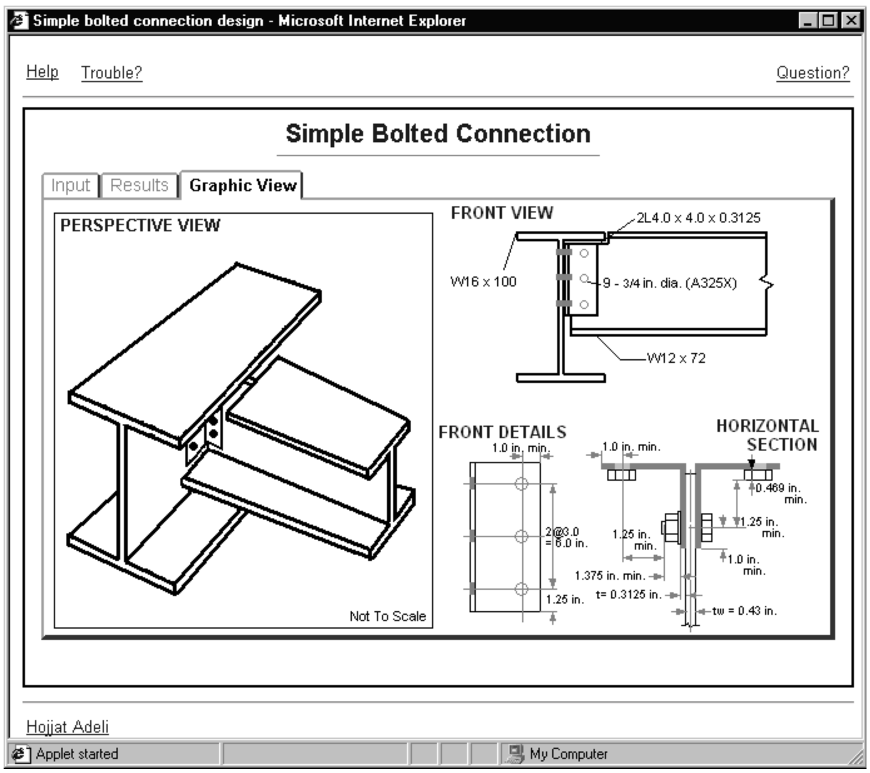


Figure 8.26 Graphic view panel for the simple bolted beam-to-girder connection design

time for downloading of the perspective view, the size of the GIF formatted file for each perspective view is limited to 3 to 4 kilobytes so that it can be transferred in less than a few seconds by the applet when the on-line connection is through a slow telephone modem. This is done by simplifying the connection perspective views and avoiding or minimizing the use of the color in displaying the perspective view.

By keeping the size of the applet relatively small the time required to download the applet itself is also reduced. Furthermore, the database of the perspective views for various connections on the server can be maintained and updated without the need to revise the source codes of the applets.

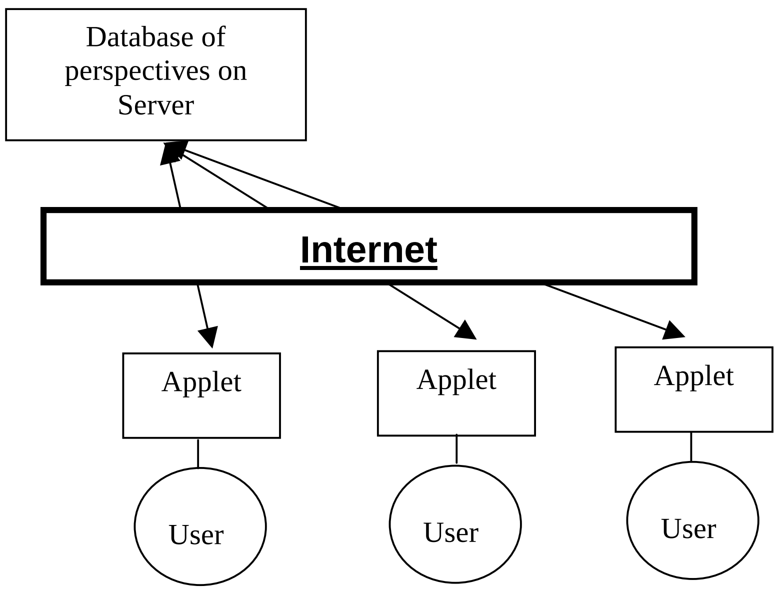


Figure 8.27 Database of connection perspectives on the server

8.4.2. Design of Simple Welded Beam-Column Connections

Similar to the applet for design of simple bolted connections the applet for design of simple welded connections also consists of three panels: *Input* panel, *Results* panel, and *Graphic View* panel.

The user enters the connection design input in the input panel (Fig. 8.28). This input includes selection of the W (wide flange) shapes

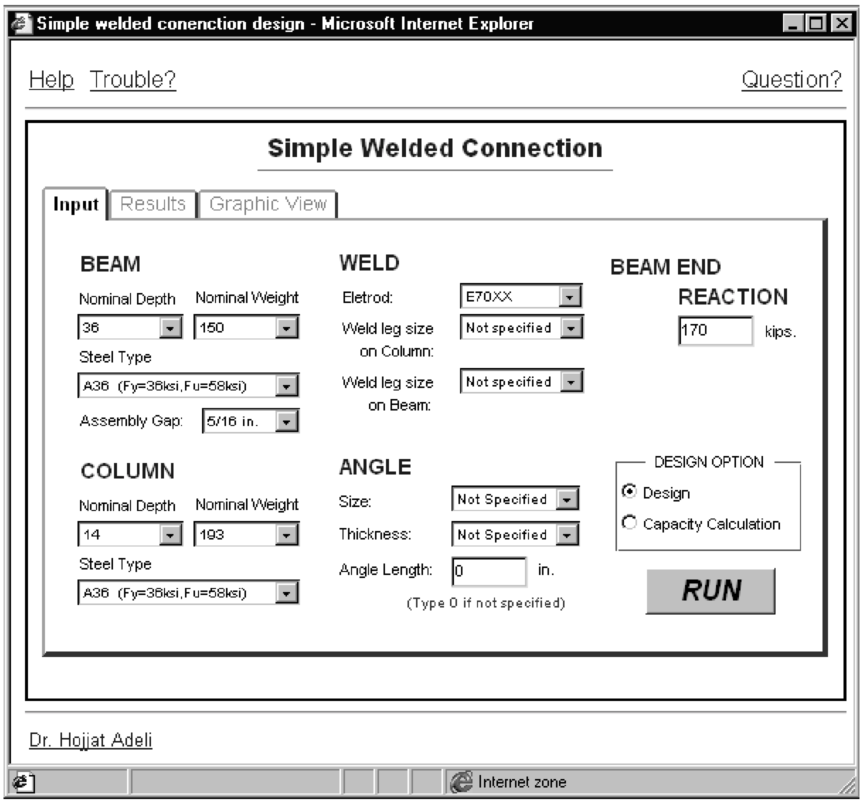


Figure 8.28 Initial/input panel for the simple welded connection design applet

for beam and column, angle, weld size and electrode, and entering the beam end reaction. The user can also choose between two options: design or compute the load-carrying capacity of the connection.

The applets accept legal practical values only. For improper or impractical input values a warning or pop-up message will be displayed and the user will be asked to reenter/re-select the input. Other bits of information and knowledge are also displayed to guide the user in the design process. As an example, when the user chooses a weld size of

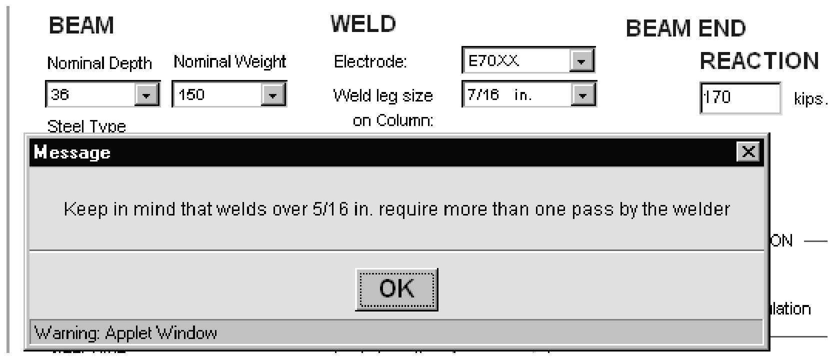


Figure 8.29 Applet warns the user when the selected weld size is greater than 5/16 in. by a pop-up message

greater than 5/16 in. the pop-up message shown in Fig. 8.29 will be displayed.

A summary of final connection design is presented in the results panel (Fig. 8.30). A perspective view of the connection and two side views of the connection with all the dimensions are displayed in the graphic view panel (Fig. 8.31). The results shown in Figs. 8.30 and 8.31 are for the example data given in Fig. 8.28.

8.5 PROBLEMS

8.1 A tension member is connected to the flange of a W shape column through a WT stub and two vertical rows of 1-in. A325 bolts, as shown in Fig. 8.32. The maximum tensile force in the member is 200 K. Find the minimum required number of bolts assuming the connection to be slip critical. Also, assume that holes are standard size, that threads are excluded from the shear plane, and that the tensile force passes through the centroid of bolts.

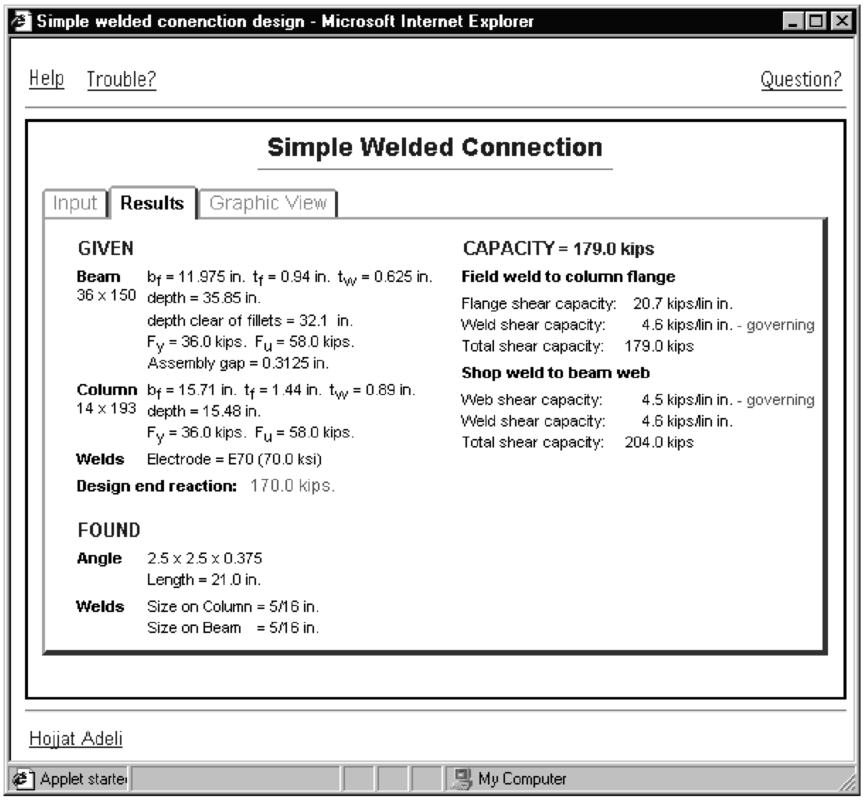


Figure 8.30 Results panel showing a summary of the simple welded connection design

8.2 A built-up beam is made of four L8x6x1 angles and two PL40x½ bolted together as shown in Fig. 8.33. Type of steel is A36 steel with yield stress of 36 ksi and an ultimate stress of 58 ksi. Using 1-in. A325 bearing-type bolts, find the spacing of the bolts along the *length* of the beam for a shear force of V = 450 K. Assume that threads are excluded from the shear planes.

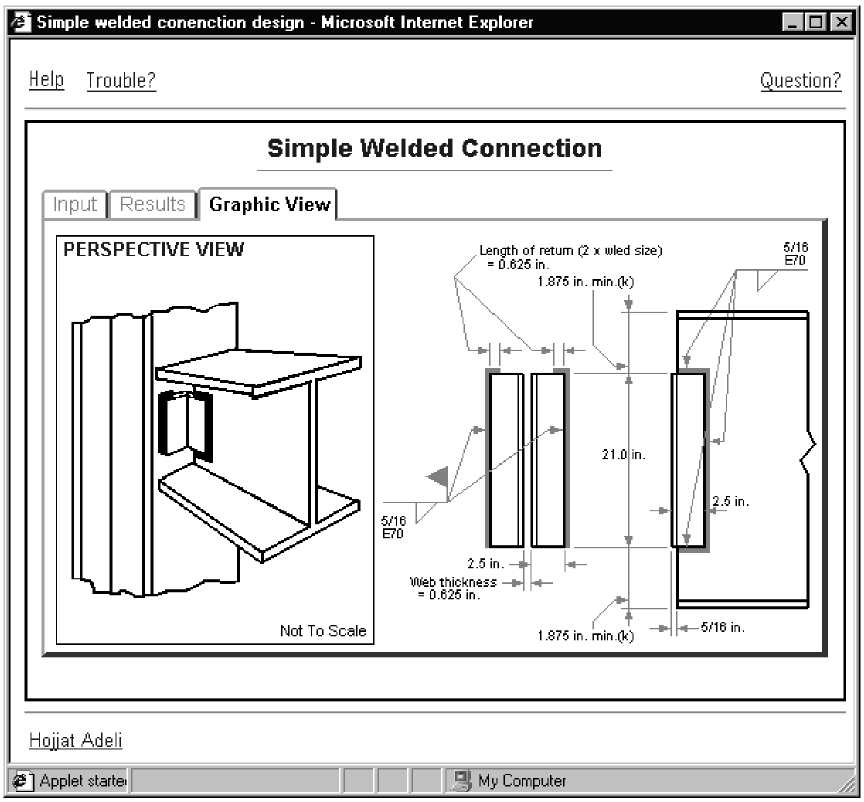


Figure 8.31 Graphic view panel for the simple welded connection design

8.3 The web of a W27x114 beam is connected to the flange of a W14x120 column through two L4x4x½ and A325 slip critical bolts, as shown in Fig. 8.34. Four 1-in.-diameter bolts are used to connect the angles to the beam web, and ten 7/8-in.-diameter bolts are used to connect the angles to the column flange. Neglecting the eccentricity, find the shear capacity of the connection. The column, the beam, and the angles are made of A36 steel with yield stress of 36 ksi.

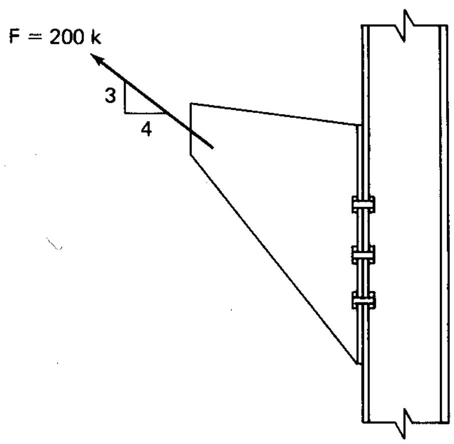


Figure 8.32

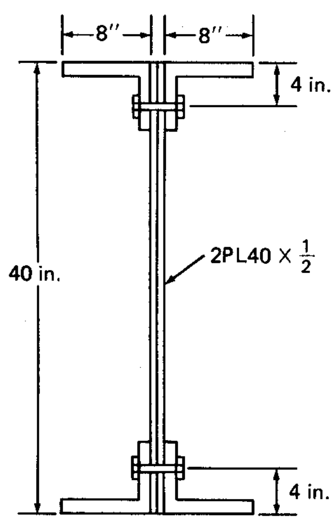


Figure 8.33

8.4 Solve Problem 8.3, using bearing-type connections with threads excluded from the shear plane.

8.5 In Problem 4.2, find the maximum tension capacity of the connection on the basis of the strength of bolts for the following cases:

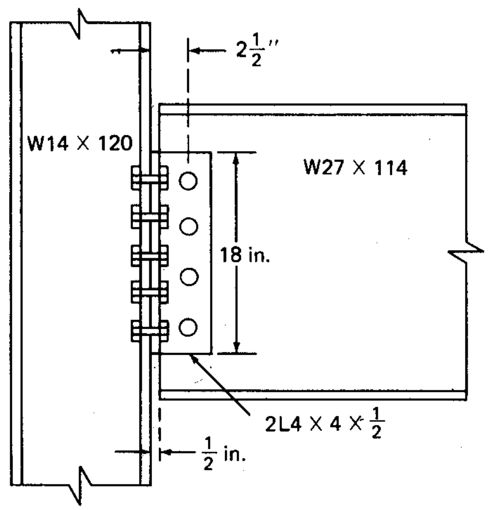


Figure 8.34

a. A325 bolts and slip critical connection
b. A325 bolts and bearing-type connection, assuming that threads are excluded from the shear planes.
c. A325 bolts and bearing-type connections, assuming that threads are not excluded from the shear plane.
d. A490 bolts and slip critical connection.
e. A490 bolts and bearing-type connection, assuming that threads are excluded from the shear planes.
f. A490 bolts and bearing-type connections, assuming that threads are not excluded from the shear plane.

8.6 Solve Example 2 of this chapter, assuming that the bottom channel is C12x25 instead of C12x30.

8.7 Solve Example 3 of this chapter, assuming that the force P = 50 K makes an angle of 45 degrees with the horizontal (to the right).

8.8 Two 1-in.-thick plates made of A36 steel with yield stress of 36 ksi are connected through fillet weld as shown in Fig. 8.35. Find the maximum force F that can be applied to the connection when ½-in. submerged arc welding and E60 electrode are used.

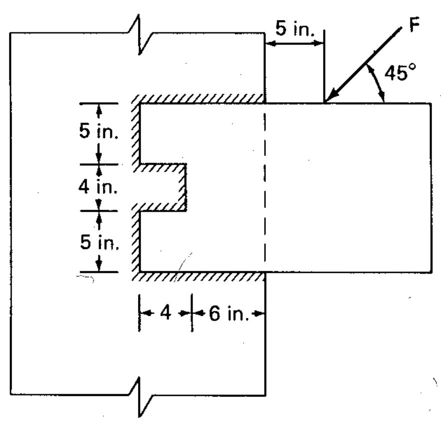


Figure 8.35

8.9 A welded built-up girder is made of a WT12x81 and a C10x30, as shown in Fig. 8.36. The maximum shear force in the girder is 100 K. Design the continuous fillet weld for connecting the two sections. Use E70 electrodes and

a. shielded metal arc welding (SMAW)
b. submerged arc welding (SAW)

8.10 Solve Problem 8.9, but use intermittent fillet weld with maximum spacing.

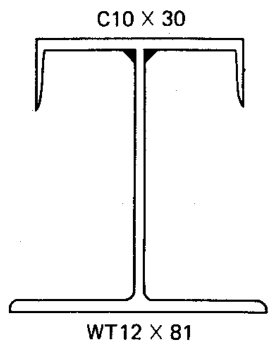


Figure 8.36