This is an unedited, uncorrected chapter.

The final chapter will be available in time for fall.

NOTE: Figures and tables appear at the end of the chapter.

WEB CHAPTER W6

Connectors

W6.1 Bolt Installation and Pretensioning Methods

The performance of a bolted joint depends on the type of bolt and upon the manner in which the bolts are tightened. Bolts in the vast majority of joints in building structures need only be tightened to what is known as the ***snug-tightened condition***, where the nuts are tightened sufficiently to prevent play in the connected members and loosening of the nut. LRFDS defines ***snug-tightened condition*** as the tightness that is attained with a few impacts of an impact wrench or the full effort of an iron worker using an ordinary spud wrench to bring the connected plies in a joint into firm contact. The initial tension developed may or may not be substantial and is often negligible. In other joints, called ***slip-critical joints*** and ***pretensioned joints***, bolts must be tightened beyond the snug-tightened condition, by one of the four methods described in Sections W6.1.1 to W6.1.4, to attain the minimum pretension specified in LRFDS Table J3.1. Tightening must progress from the most rigid part of the joint to its free edges.

W6.1.1 Turn-of-Nut Pretensioning

The theory behind the ***turn-of-nut pretensioning method*** is that a specific rotation of the nut, beyond the snug-tightened condition, causes a specific amount of strain in the bolt which corresponds to a specific pretensioning force. Thus, for a given type, length, and size of bolt, the partial turn of the nut required to produce a given pretension can be calculated. For example, a ¾ in. dia. 10 threads-per-inch A325 bolt with a grip of 4 in., corresponding to the ⅓ turn specified, will undergo a strain of $(\frac{1\text{ in.}}{10} \times \frac{1}{3}) \div 4\text{ in.} = 0.008\text{ in./in.}$ This is far greater than the maximum elastic strain of the bolt material : $\frac{F_{yb}}{E} = \frac{92}{29{,}000} = 0.003\text{ in./in.}$ Of course there is some compression of the material being bolted and some galling of the bearing surfaces, but there is little doubt that high preloads are produced in the bolts by the turn-of-nut pretensioning method.

In the turn-of-nut pretensioning method, bolts are installed in all holes of the joint and are brought to a snug-tightened condition. From the snug-tightened position, all bolts in the joint are tightened further by giving an additional 1/3 to 1 turn, the amount depending upon the length of the bolt as specified in Table 8.2 of the RCSCS [2000]. The turn that is applied is relative between the bolt and the nut; thus the element not turned in tightening should be prevented from rotating while the required degree of turn is applied to the turned element. When bolts are tightened by the turn-of-nut pretensioning method, the outer face of the nut may be match-marked with the protruding end of the bolt after snug tightening, thus presenting the inspector with a visual means of noting nut rotation. Such marks may be made with crayon or paint by the wrench operator after the bolts have been snug-tightened.

W6.1.2 Direct-Tension-Indicator Pretensioning

Figure W6.1.1: Direct tension indicator.

The ***ASTM F959 direct tension indicator*** (DTI) is a hardened, circular, washer-shaped device incorporating several small arch-like protrusions pressed out of the flat surface (Fig. W6.1.1). The washer is generally interposed between the bolt head and the gripped material with the protrusions bearing against the hardened bearing surface of the bolt head leaving a gap. Upon tightening, the bolt pretension causes the protrusions to yield plastically and partially flatten, reducing the gap between the washer's flat surface and the bolt head. Bolt tension is evaluated from measurements of the residual gap with the help of a feeler gage. In order to obtain the specified minimum bolt pretension, the manufacturer recommends tightening until this gap is reduced to 0.015 in. or less. Load indicator washers are the single device available which are directly dependent upon the tension load in the bolt, rather than upon some indirect parameter, to indicate the tension in a bolt.

W6.1.3 Twist-Off-Type Tension-Control Bolt Pretensioning

Figure W6.1.2: Twist-off-type tension-control bolt.

Twist-off-type tension-control bolts meet the requirements of ASTM F1852. These bolts have a ***splined end*** that extends beyond the threaded portion of the bolt, a design feature intended to help automatically provide the required bolt pretension (Fig. W6.1.2). Between the splined end and the eventual end of the bolt is an annular groove that is designed to shear at a predetermined torque during installation. The heads of these bolts are hexagonal similar to a standard bolt, or rounded, similar to a rivet head. The bolts are produced and shipped by the manufacturer as a bolt-nut-washer assembly. The specially designed wrench used to pretension these bolts is provided with two sockets which counter rotate: an inner socket for the splined end of the bolt and an outer socket for the nut. The calibrated bolt tip twist-shears from the bolt when the proper bolt tension is reached. The sheared tip will remain in the inner socket until ejected by actuating the ejector rod.

The non-impacting electric wrench used eliminates need for air compressors and operates at lower noise levels. The electric gun is lighter than a pneumatic wrench. One-side, one-man installation maximizes productivity. The fact that this bolt provides the inspector with an easy way of verifying that the minimum required torque has been applied makes this a popular bolt in steel structures.

W6.1.4 Calibrated Wrench Pretensioning

In the ***calibrated wrench pretensioning*** method an air-operated torque wrench, which is calibrated or adjusted to "stall" at a preset torque, is used to pretension the bolts. When the calibrated wrench pretensioning method is used, a hardened washer must be used under the turned element.

Data on bolt clamping force indicates that bolt pretensions are distributed normally for each pretensioning method. However, the data also indicates that the mean value of the bolt pretension is different for each method. If the calibrated wrench method is used to pretension A325 bolts, the mean value of bolt pretension is about 1.13 times the specified minimum pretension, T_b (LRFDS Table J3-1). For the turn-of-nut pretensioning method, the mean value of pretension is about 1.35 times the specified minimum pretension for A325 bolts and about 1.26 for A490 bolts.

W6.2 Classification of Bolted Joints

Based on connection performance two broad categories of bolted joints are provided for in the LRFD Specifications; namely, ***slip-critical joints*** and ***bearing-type joints***. Bearing-type joints may be further subdivided into ***fully-pretensioned bearing-type joints*** (or, simply, ***pretensioned joints***), and ***snug-tightened bearing-type joints*** (or, simply, ***snug-tightened joints***).

Slip-critical joints are those joints that have specified faying surface conditions that, in the presence of clamping provided by pretensioned bolts, resist a design load in the plane of the joint solely by friction and without displacement (slip) at the faying surfaces. Slip-critical joints, therefore, have a low probability of slip at anytime during the life of the structure.

In a ***bearing-type joint***, slip is acceptable, and shear and bearing actually occur. Under the provisions of LRFDS, certain bearing- type joints are required to be pretensioned, but are not required to be slip-critical. Such joints are known as ***pretensioned joints***.

All other joints are ***snug-tightened joints***.

W6.2.1 Slip-critical joints

Bolts in ***slip-critical joints*** are pretensioned by one of the four methods described in Section W6.1, so as to develop a specified minimum pretension in the bolt. The ***specified minimum bolt pretension*** for high-strength bolts is equal to 70% of the specified minimum tensile strength of the bolt, rounded to the nearest kip. Thus, we have:

$$T_b = 0.7\, F_{ub}\, A_b \tag{W6.2.1}$$

where T_b = specified minimum bolt pretension, kips

A_b = area of bolt shank, in.^{2}

F_{ub} = minimum specified ultimate tensile stress of the bolt material (Eq. 6.2.1), ksi

Numerical values of the specified minimum bolt pretension loads, T_b, are given in LRFDS Table J3.1 for various bolt diameters, for both A325 and A490 bolts (reproduced partially in Table 6.7.1). Note that T_b nearly equals the proof load for A325 bolts, and is about 85 to 90% of the proof load for A490 bolts. For example, the pretension load specified for a ¾-in.

dia. A325 bolt is 28 kips. It is this high pretension load that prevents the nuts from working loose even under vibratory loading. The high installation force introduces high localized contact pressures. Studies have shown that the contact stresses between faying surfaces are mainly concentrated in a region equal to about 2 to 3 diameters from the bolt hole.

In a slip-critical joint, the fully-pretensioned bolt creates resistance to slip through the friction on the faying surface between two connected parts. This slip resistance, as seen from Eq. 6.4.1, is a function of the slip coefficient μ of the faying surface. RCSC Specification [RCSC, 2000] defines three classes of surface preparation. Thus it defines, unpainted clean mill scale steel faying surfaces (or surfaces with Class A coatings on blast-cleaned steel) as Class A surfaces with $\mu = 0.33$; unpainted blast-cleaned faying surfaces (or surfaces with Class B coating on blast-cleaned steel) as Class B surfaces with $\mu = 0.50$; and hot-dip galvanized and roughened surfaces as Class C surfaces with $\mu = 0.35$. Slip coefficients for all other coated blast-cleaned surfaces should be determined by the procedures given in Appendix A of the RCSC Specification. When the tests result in $0.33 \leq \mu < 0.50$, the coating is a Class A coating and the design slip coefficient $\mu = 0.33$. If the tests result in $\mu \geq 0.50$, the coating is a Class B coating and the design slip coefficient $\mu = 0.50$. It is important to remember that the surface requirements for slip-critical joints apply only to the faying surfaces, and do not include the surfaces under the bolt, washer, or nut.

Slip-critical joints are required in the following situations:

1. Joints that are subject to fatigue load with reversal of the loading direction must be slip-critical since slip may result in back-and-forth movement of the joint and the potential for accelerated fatigue failure. This is primarily of concern for connections in crane girders, structures supporting machinery, equipment, and in bridges.
2. Joints in which bolts and welds are assumed to share in load transfer at a common faying surface (LRFDS Section J1.9).
3. Joints with bolts installed in oversized holes (LRFDS Section J3.2).
4. Joints with bolts installed in slotted holes where the applied load on the joint is in a direction other than normal (between 80° and 100°) to the direction of the long dimension of the slot (LRFDS Section J3.2). Unless slip is intended, as in a sliding expansion joint, slip in joints with long-slotted holes that are parallel to the direction of the applied load might be large enough to invalidate structural analyses that are based upon the assumption of small displacements.
5. The consequences of slip are not limited to strength and stability. Serviceability limit states, such as excessive deflection, may also dictate that joint slippage be prevented.
6. Connectors between elements of bolted built-up compression members at their end connections, so that the individual components will act as a unit in column buckling (LRFDS Sections E4.2). In effect, in built-up compression members such as double angle struts in trusses, a small relative slip between the elements especially at the end connections can increase the effective length of the built-up section to that of the individual components and significantly reduce the compressive strength of the

member. Section E4.2 of the LRFDS specifies that "*the end connections shall be ... fully-pretensioned bolted with clean mill scale or blast-cleaned faying surfaces with Class A coatings.*" That means, the end connection can be proportioned as a bearing connection as long as the faying surfaces offer at least Class A slip coefficient.

W6.2.2 Pretensioned joints

The specified minimum bolt pretension, T_b, for bolts in pretensioned joints is the same as that required for bolts in slip-critical joints (given by Eq. W6.2.1 and tabulated in LRFDS Table J3.1). Pretensioned joints are required in the following situations:

1. When bolts in bearing-type joints using A490 high-strength bolts are subjected to tension only or combined shear and tension under external loads, with or without fatigue, these bolts must be fully-pretensioned to T_b. Some A490 high strength bolt applications where fully-pretensioned bearing-type joints need to be provided, are:
- hanger connections (Fig. 6.1.2*d*),
- diagonal bracing connections (Fig. 6.1.2*e*), and
- extended end-plate FR moment connections.

2. Joints with A325 or F1852 bolts that are subject to tensile fatigue and joints that are subject to fatigue load with no reversal of the loading direction must be fully-pretensioned to T_b.

3. Further, pretensioned A325 or A490 high-strength bolts (or welds) must be used for the following shear connections:

- Column splices in all tier structures over 125 ft.
- In structures with $H >$ 125 ft, those beam-to-column connections, girder-to-column connections, and any connections of members that provide bracing to columns.
- In all building structures carrying cranes over 5 ton capacity
  - crane supports,
  - connections of trusses to columns
  - column splices,
  - roof truss splices
  - column bracing connections.
- Connections for the support of machinery, and other live loads that produce impact or stress reversal.

W6.2.3 Snug-tightened joints

When pretension is required to prevent slip, a slip-critical joint should be specified. When pretension is desired for reasons other than the necessity to prevent slip, a pretensioned joint should be specified. In all other joints, the bolts could be ordinary bolts (A307), or high-strength bolts (A325, F1852, or A490), and the bolts need only be snug-tightened. These joints are known as ***snug-tightened bearing-type joints*** or, simply, ***snug-tightened joints***. Snug-tightening the bolts induces only small clamping forces in the bolts (less than 30%T_b). Also, these forces can vary considerably because elongations are still within the elastic range. So, no frictional resistance on the faying surfaces is assumed and for design purposes slip in bearing-type joints is assumed to occur as soon as external loads are applied.

The following comments should be taken into account in specifying the type of joints:

- Current inspection requirements for snug-tightened joints consist of verification that proper fastener components were used and of checking the nut visually for evidence of impacting with an impact wrench [Section 9.1 of the RCSC specification, RCSC, 2000]. On the other hand, each of the four methods of installation described in the Section 6.5 to pretension the bolts requires special tools, considerable effort on the part of iron worker, and careful and costly inspection [Section 9.2 of the RCSC Specification, RCSC, 2000]. Thus, pretensioned joints should be specified sparingly. When joints are designed as pretensioned, they are not subject to the same faying surface treatment and inspection requirements as is required for slip-critical joints. Thus, painted surfaces in pretensioned joints need not meet the stringent slip-resistance requirements of slip-critical joints. Further, a joint designed as slip-critical requires more bolts than when designed as a bearing-type joint. Thus, slip-critical joints should be used only where absolutely necessary.
- As discussed earlier, load applications sufficient to cause a joint to repeatedly slip in one direction and then the other are those for which the joint must be designed as slip-critical. LRFDS Section K3 states that: "*The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design...*" Consequently, slip-critical joints are not normally required or used for wind or seismic loading in buildings. However, a member that vibrates due to even a steady wind may require slip-critical joints.

Figure W6.2.1: Force transfer mechanisms in bolted joints.

Force transfer mechanisms for bolts in slip-critical, snug-tightened, and pretensioned joints are shown in Fig. W6.2.1. As shown in Fig. 6.4.2, up until the slip load is reached all pretensioned high-strength bolted joints resist load in the plane of the joint by friction between the faying surfaces. Until slip occurs, therefore, there are no shearing or bearing forces on the bolt; the bolt is loaded by the pure axial tension created when the nuts were tightened. They subsequently resist even greater loads by shear and bearing. The onset of slipping in a high-strength bolted, slip-critical joint is not an indication that maximum capacity of the joint has been reached. Its occurrence may be only a serviceability limit state.

W6.3 Behavior of a Single Bolt in Shear (Bearing-type joint)

W6.3.1 Load-Deformation Response

Figure W6.3.1: Compression jig for testing a single bolt in shear.

Shear load versus deformation relationships for bolts are generally obtained by subjecting bolts to double shear induced by plates either in compression or tension (a compression jig is shown schematically in Fig. W6.3.1). The deformations measured were the sum of the shearing deformation of the bolt and the bearing deformations of the bolt and adjacent plates.

Figure W6.3.2: Load-deformation response of bolts in shear.

Based on the results of six single bolt tests, using ¾-in. diameter A325 bolts installed in 4 in. ×4 in. A36 steel plates and loaded in compression, Crawford and Kulak [1971] presented a theoretical load-deformation curve which best fits the experimental data (Fig. W6.3.2). For small values of deformation this relationship is approximately linear and as the deformation approaches ultimate strength, the bolt force increases at a decreasing rate. This load-deformation relationship of a single bolt, in a bearing-type connection, can be expressed as:

$$B_v = B_{uv}[1 - e^{-\mu\Delta}]^{\lambda} \tag{W6.3.1}$$

where B_{uv} = ultimate shear strength of a high-strength bolt, kips

Δ = total deformation of the bolt, including shearing, bending, and bearing deformations, plus local bearing deformation of the plates, in.

B_v = shear force in the bolt at deformation Δ, kips

e = base of the natural logarithm (= 2.718 . . .)

μ, λ = regression coefficients

For the ¾-in. dia. A325 bolts tested, the mean maximum bolt force was 74.0 kips with a standard deviation of 2.4 kips and the mean maximum deformation was 0.34 in. with a standard deviation of 0.03 in. The values of μ and λ are 10.0 and 0.55 respectively. Thus

$$B_{uv} = 74.0 \text{ kips}\,; \qquad \Delta_{\max} = 0.34 \text{ in.} \qquad \text{(W6.3.2)}$$

$$\mu = 10.0\,; \qquad \lambda = 0.55$$

Measurements of the internal tension in bolts in joints in shear have shown that at ultimate load there is little pretension left in the bolt. The shearing deformations that have taken place in the bolt prior to its failure release the rather small amount of axial deformation that was used to induce the bolt pretension during installation. Test results on high-strength bolts torqued to "snug", ½ turn, and 1½ turns of the nut from the snug position, and then loaded in direct shear, indicate that pretension has little influence on the ultimate shear strength of the bolt [Wallaert and Fisher, 1965; Bendigo et al., 1963; Fisher, Ramseier, and Beedle , 1963]. Also, as mentioned earlier, Section 5.1 of the RCSCS stipulates that the design shear strength of a bolt shall not be reduced by the installed bolt pretension.

W6.3.2 Bearing Stress Distribution

Under zero load, contact between a cylindrical bar (bolt) and the interior of an encircling cylinder of larger diameter would be, theoretically, along a single line. If both the bar and the encircling cylinder are very long, and the material perfectly elastic, the contact stress could be calculated using Hertz's theory and is given by [Roark, 1965]:

$$f_{b\max} = 0.798 \sqrt{\frac{q\,E}{2(1 - \mu^2)} \frac{(d_2 - d_1)}{d_1 d_2}} \qquad \text{(W6.3.3)}$$

Here $f_{b\,\max}$ is the maximum bearing pressure; q, the load per unit length of cylinder; d_1, the diameter of the bar; d_2, the diameter of the hole; μ, the Poisson's ratio and E, the modulus of elasticity. For steel with modulus of elasticity, $E = 29{,}000$ ksi and Poisson's ratio, $\mu = 0.3$, the above relation reduces to:

$$f_{b\max} = 100 \sqrt{\frac{q\,(d_2 - d_1)}{d_1 d_2}} \qquad \text{(W6.3.4)}$$

where q is in kli, d_1 and d_2 are in inches, and $f_{b\,\max}$ is in ksi. Although the relation should be modified before applying to pieces of finite length, such as a bolt in a thin plate, an idea of the order of magnitude of the computed bearing stress may be obtained by letting: $d_2 - d_1 = 1/16$ in., which is the clearance for a bolt in a standard bolt hole, $d_1 = 1$ in., and $q = 104$ kli (which corresponds to the design bearing strength of a 1-in. dia. bolt in standard hole in 1 in. thick A36 plate with $F_y = 36$ ksi and $F_u = 58$ ksi, from LRFDM Table 7-13), as:

$$f_{b\max} = 100 \sqrt{\frac{104\,(1/16)}{1.00\,(17/16)}} = 247 \text{ ksi}$$

The high value of the bearing stress obtained indicates that bearing stresses lie in the yield range. Thus, as the load is applied and increased, the bearing stresses are initially concentrated at the points of contact. An increase in load causes local yielding and a larger area of contact, together with a more uniform distribution of the compressive stress.

Figure W6.3.3: Bearing stress distribution in bolted connections.

When a bolted joint is loaded, after slip occurs the bolt shank bears against the side of the hole. Bearing (or contact) pressure develops in the plate material adjacent to the hole and in the bolt. The actual stress distribution, in the elastic domain, is even more complicated than that obtained for the idealized case discussed above. Figure W6.3.3 shows such distributions for a bolt in double shear and for a bolt in single shear. We observe that, the contact stresses over the height of the cover plates of a butt joint is less uniform than on the main plate. In a single shear lap joint the bearing stress distribution is even more complex (Fig. W6.3.3*a*). However, with ductile plate materials, at high load levels, elastic and then plastic deformations cause widening of the contact zone and this results in the more uniform stress distribution shown in Fig. W6.3.3*b*.

Although the bolt itself is subject to the same magnitude of compressive forces as those acting on the side of the hole, tests have shown that the bolt is not critical. Thus, the same bearing value applies to joints assembled by all high-strength bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area. Also, the nominal bearing strength per unit projected area at a bolt hole is the same for double shear bearing and single shear bearing. While there is a difference in the stress distribution in the two cases (Fig. W6.3.3*a*), particularly at low load levels, tests have shown that this difference is reduced by plastic redistribution to the extent where it has no apparent effect on the ultimate behavior.

W6.4 Single Bolt in Combined Shear and Tension (Bearing-type Joint)

W6.4.1 Bolt Behavior in Combined Shear and Tension

Bolts in wind bracing connections (Fig. 6.1.2*e*) are often subjected to both shear and tension under applied loads. Also, certain bolts in seated beam connections and beam-to-column connections (discussed in Chapter 13) are also subjected to combined shear and tension under external loads. The possible reduction in the strength of bolts when they are simultaneously subjected to shear and tension under external loads, has to be considered in the design. The LRFD specification requires in Section J3.1 that A490 high-strength bolts in such connections, subjected to tension under external loads shall be pretensioned to a bolt tension T_b specified in LRFDS Table J3.1.

Bolts in shear in pretensioned bearing-type joints and bolts in shear in all slip-critical joints are actually under tensile stresses as well, because of the existence of the pretension. These bolts, however, are never checked for combined stresses because past experience shows that bolts designed separately for shear and bearing will usually withstand the high combined stresses generated due to the presence of any pretension. When bolts are subjected simultaneously to shear and tension under external loads, the usual practice is to consider the combined effects in the design process. In addition to tensile and shearing stresses, the bolts are also subjected to bearing stresses, although not every portion of the bolt is subjected to these three stresses to the same degree. The present practice is to neglect the bearing stresses in evaluating the effect of combined stresses. Thus, for example, the LRFD Specification for bolts in bearing-type joints limits combined tension and shear loads by an interaction equation that considers nominal tensile and shear stresses only and neglects initial tension, friction, and bearing. Tests performed at the University of Illinois by Chesson, Faustino and Munse [1965] form the basis of the interaction formulae. In these tests, the strength and behavior of a single high-strength bolt subjected to various ratios of applied tension and shear forces were studied. A total of 115 bolts were tested. The parameters studied were: types of high strength bolts (A325 and A354 Grade BD, nearly identical to A490); bolt diameters (¾ in. and 1 in.); bolt grip length (3 in. to 6 in.); type of shear plane (N-type, X-type); joint material, etc. It was concluded that neither bolt diameter, nor the joint material had significant effect on the ultimate capacity of the bolt. An elliptical interaction curve was found to provide a good representation of the experimental points [Chesson et al., 1965], namely

$$\left(\frac{f_t}{F_{ut}}\right)^2 + \left(\frac{f_v}{0.62\,F_{ut}}\right)^2 = 1.0 \qquad \text{(W6.4.1)}$$

Here, the tensile strength F_{ut} of the bolt material is used to non-dimensionalize the simultaneous shear stress, f_v, and tensile stress, f_t, due to the shear and tensile components of the load acting on the bolt. The tensile stress was computed on the basis of the stress area, whereas the shear stress is dependent on the location of the shear plane (X-type or N-type bolt).

W6.4.2 Bolt Strength in Combined Shear and Tension

The strength interaction relation for combined shear and applied tension, for high-strength bolts in bearing-type joints, may be represented by the elliptical relationship [Eq. 5.2 of the RCSC Specification, RCSC, 2000]:

$$\left(\frac{B_{tu}}{B_{dt}}\right)^2 + \left(\frac{B_{vu}}{B_{dv}}\right)^2 \leq 1 \qquad \text{(W6.4.2)}$$

where B_{tu} = tensile component of applied factored load for combined shear and tension loading

B_{vu} = shear component of applied factored load for combined shear and tension loading

B_{dt} = design strength of the bolt in tension, when the bolt is subjected to tension only

B_{dv} = design strength of the bolt in shear, when the bolt is subjected to shear only

The design shear strength of a high-strength bolt B_{dv}, is given by Eq. 6.7.3 and the design tensile strength of a high-strength bolt, B_{dt}, is given by Eq. 6.9.3. Equation W6.4.2 could be rewritten as:

$$B_{tu} \leq \sqrt{B_{dt}^2 - \left(\frac{B_{dt}}{B_{dv}}\right)^2 B_{vu}^2} \qquad \text{(W6.4.3)}$$

Or, equivalently

$$B_{tu} \leq B'_{dt} \qquad \text{(W6.4.4}a\text{)}$$

with

$$B'_{dt} = \sqrt{B_{dt}^2 - \left(\frac{B_{dt}}{B_{dv}}\right)^2 B_{vu}^2} \qquad \text{(W6.4.4}b\text{)}$$

Dividing both sides of the relations W6.4.4*a* and W6.4.4*b* by A_b, noting that $B_{vu} = A_b N_s f_{vu}$, where f_{vu} is the shear stress in the bolt produced by the factored loads, we obtain with the help of Eqs. 6.7.3 and 6.9.3:

$$f_{tu} \leq F'_{dt} = \phi F'_{nt} \qquad \text{(W6.4.5}a\text{)}$$

$$\begin{aligned} F'_{nt} &= \sqrt{(0.75 F_{ub})^2 - 6.25 f_{vu}^2} \quad \text{for N-type bolts} \\ &= \sqrt{(0.75 F_{ub})^2 - 4.00 f_{vu}^2} \quad \text{for X-type bolts} \end{aligned} \qquad \text{(W6.4.5}b,c\text{)}$$

Here, F'_{nt} is used instead of F_{nt}, to signify that the nominal tensile strength per unit area is now in the presence of a simultaneously acting shear stress f_{vu} computed at factored-load level. More specifically, using F_{ub} = 120 ksi and 150 ksi for A325 and A490 bolts, respectively, we obtain the elliptic relations given in LRFDS Table A-J3.1 for nominal tensile stress on bolts in bearing type joints, in the presence of simultaneously acting shear stresses, namely:

$$
\begin{aligned}
F'_{nt} &= \sqrt{90^2 - 6.25 f_{vu}^2} && \text{for A325-N Type bolts} \\
&= \sqrt{90^2 - 4.00 f_{vu}^2} && \text{for A325-X Type bolts} \\
&= \sqrt{113^2 - 6.25 f_{vu}^2} && \text{for A490-N Type bolts} \\
&= \sqrt{113^2 - 4.00 f_{vu}^2} && \text{for A490-X Type bolts}
\end{aligned}
\tag{W6.4.6}
$$

The elliptic relation, W6.4.2, can be replaced, with only minor modifications, by three linear relations, namely:

$$
\frac{B_{tu}}{B_{dt}} + \frac{B_{vu}}{B_{dv}} \leq C ; \quad \frac{B_{tu}}{B_{dt}} \leq 1.0 ; \quad \frac{B_{vu}}{B_{dv}} \leq 1.0 \qquad \text{(W6.4.7}a, b, c\text{)}
$$

where C is a constant, taken as 1.30 by the LRFDS. This linear representation is more conservative than the elliptical curve over much of its length. This representation, also shown in Fig. 6.10.1, offers the advantage that no modification of either type of load is required in the presence of fairly large magnitudes of the other type of load. The linear interaction relation W6.4.7a can be rewritten as:

$$B_{tu} \leq B'_{dt} \equiv \phi B'_{nt} \qquad (W6.4.8)$$

Here, B'_{dt} represents the design strength of a bolt in tension when the bolt is subjected to combined shear and tension, and B'_{nt} its nominal strength. We have:

$$B'_{dt} = c_1 - c_2 B_{vu} \leq B_{dt} \qquad \text{for } B_{vu} \leq B_{dv} \qquad (W6.4.9)$$

where the coefficients c_1 and c_2 are given by:

$$c_1 = 1.30\, B_{dt}; \quad c_2 = \frac{B_{dt}}{B_{dv}} \qquad (W6.4.10)$$

Further,

$$B'_{nt} = C_1 - C_2 B_{vu} \leq B_{nt} \qquad \text{for} \quad B_{vu} \leq B_{dv} \qquad (W6.4.11)$$

where

$$C_1 = c_1 \div \phi; \quad C_2 = c_2 \div \phi \qquad (W6.4.12)$$

Values of B_{dt} and B_{dv} for different types of high-strength bolts are given in Table 6.7.1.

The linear interaction relation W6.4.7*a* could also be written in stress format by dividing both sides by the nominal bolt area, A_b, and using Eqs. 6.7.3 and 6.9.3. First, considering N-type bolts:

$$\frac{B_{tu}}{A_b} = f_{tu} \leq 1.30\,\phi(0.75\,F_{ub}) - \frac{\phi(0.75\,F_{ub})B_{vu}}{0.75\,(0.4\,F_{ub})\,N_s A_b}$$

or, equivalently, by:

$$f_{tu} \leq \phi F'_{nt} \tag{W6.4.13a}$$

with

$$F'_{nt} = 0.975\,F_{ub} - 2.5 f_{vu} \tag{W6.4.13b}$$

By substituting the value of F_{ub} = 120 ksi for A325 bolts considered, Eq. W6.4.13*b* becomes:

$$F'_{nt} = 117 - 2.5 f_{vu} \tag{W6.4.13c}$$

The tri-linear representation of Eq. W6.4.7, in stress format, may now be written as:

$$f_{tu} \leq \phi F'_{nt} \leq \phi F_{nt}\,; \qquad f_{vu} \leq \phi F_{nv} \tag{W6.4.14}$$

As mentioned earlier values of ϕ, F_{nv} and F_{nt} are given in LRFDS Table J3.2.

By proceeding similarly, we obtain the linear interaction formulae for combined shear and tension, in stress format, for high-strength bolts as (LRFDS Table J3.5):

$$F'_{nt} = 117 - 2.5 f_{vu} \leq 90 \quad \text{for } f_{vu} \leq 36 \text{ ksi, for A325-N type} \qquad \text{(W6.4.15}a\text{)}$$

$$= 117 - 2.0 f_{vu} \leq 90 \quad \text{for } f_{vu} \leq 45 \text{ ksi, for A325-X type} \qquad \text{(W6.4.15}b\text{)}$$

$$= 147 - 2.5 f_{vu} \leq 113 \quad \text{for } f_{vu} \leq 45 \text{ ksi, for A490-N type} \qquad \text{(W6.4.15}c\text{)}$$

$$= 147 - 2.0 f_{vu} \leq 113 \quad \text{for } f_{vu} \leq 56 \text{ ksi, for A490-X type} \qquad \text{(W6.4.15}d\text{)}$$

Here f_{vu} is the shearing stress in the bolt resulting from the factored loads (= $B_{vu}/(N_s A_b)$). A graphical representation of these four relations is shown in Fig. 6.10.2.

W6.5 Slip Resistance of a Bolt (Slip-Critical Joint)

Slip-critical joints give very good service under impact, vibratory loading, or loading involving stress fluctuation or reversal. The first limit state for a slip-critical joint is the occurrence of slip. Until slip occurs the bolts are not actually stressed either in shear or in bearing. Thus, under the loads corresponding to this limit state there isn't a true shear failure across the shank of the bolt. Nevertheless, it is common practice and a design expedient to specify the friction capacities as a design shear stress on the nominal area of the bolt for slip-critical joints as well, in order that the proportioning of slip-critical joints may be carried out using the same familiar methods of bearing-type joint design. Of course, the design resistance to shear a bolt in a slip-critical joint is lower than that for a similar bolt in a bearing-type joint. Slip of connections designed as slip-critical is likely to occur at approximately 1.4 to 1.5 times the service loads.

LRFDS permits the design of high-strength bolts in slip-critical joints to be made either at the factored-load level using LRFDS Section J3.8*a*, or at the service-load level using LRFDS Appendix J3.8*b*. The corresponding slip resistances are designated in this text as B_{nsf} and B_{nss}, respectively.

W6.5.1 Design Slip Resistance at the Factored-Load Level

The design slip resistance of a bolt in a slip-critical joint, for design at the factored-load level may be written (with the help of LRFDS Eq. J3-1) as:

$$B_{dsf} \equiv \phi_s B_{nsf} = \phi_s (1.13) \mu T_b N_s \qquad \text{(W6.5.1}a\text{)}$$

where B_{dsf} = design slip resistance of a bolt in a slip-critical joint, for design at the factored-load level, kips

B_{nsf} = nominal slip resistance of a bolt in a slip-critical joint, for design at the factored-load level, kips

T_b = specified minimum bolt pretension given in LRFDS Table J3.1 (Table 6.7.1), kips

N_s = number of slip planes

ϕ_s	=	resistance factor	
	=	1.0	for STD holes
	=	0.85	for OVS and SSL holes
	=	0.70	for LSL holes transverse to the direction of load
	=	0.60	for LSL holes parallel to the direction of load
μ	=	mean slip coefficient (coefficient of static friction)	
	=	0.33	for Class A faying surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel)
	=	0.50	for Class B faying surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)
	=	0.35	for Class C faying surfaces (roughened hot-dip galvanized surfaces)

For example, the design slip resistance for an A325-SC bolt in single shear, in a standard hole, and Class A surface preparation is given by Eq. W6.5.1*a* as: 1.00 (1.13) $(0.33T_b)(1) = 0.373T_b$ kips. Because of the greater concern for the consequences of a slip of significant magnitude if it should occur in joints with oversized holes, or in the direction of the slot for slotted holes, lower values of design slip resistance are provided for joints with these hole types through a reduction of the resistance factor ϕ. Further, for joints with long-slotted holes, even though the slip load is the same for loading transverse or parallel to the axis of the slot, the design value for loading parallel to the axis has been further reduced, based upon judgment in recognition of the greater consequences of slip.

The design slip resistance must equal or exceed the shear force on the bolt, under factored loads; that is:

$$B_{dsf} \geq B_{vu} \qquad \text{(W6.5.1}b\text{)}$$

where B_{vu} = shear force on a bolt, using factored loads, kips

W6.5.2 Design Slip Resistance at the Service-Load Level

The design slip resistance of a bolt in a slip-critical joint, designed at the service-load level, from RCSCS Section 5.4.2, is:

$$B_{dss} \equiv \phi_s B_{nss} = \phi_s (0.8) \mu T_b N_s \qquad \text{(W6.5.2)}$$

where B_{dss} = design slip resistance of a bolt in a slip-critical joint, for design at the service-load level

B_{nss} = nominal slip resistance of a bolt in a slip-critical joint, for design at the service-load level and the resistance factor, ϕ_s; slip coefficient, μ; bolt pretension, T_b; and number of slip surfaces, N_s, are as defined in Eq. W6.5.1*a*. LRFDS Appendix J3.8*b* expresses B_{dss} in a slightly different format, namely:

$$B_{dss} = \phi_{ss} F_v A_b N_s \qquad \text{(W6.5.3}a\text{)}$$

where ϕ_{ss} = resistance factor (= 1.0)

A_b = nominal area of bolt, in.^{2}

F_v = nominal slip-critical shear resistance per unit area of bolt (tabulated in LRFDS Table A-J3.2), ksi

= ϕ_s (0.8) μ 0.75 F_{ub}

N_s = number of slip planes

The nominal slip-critical shear resistance, F_v, is a function of the coefficient of static friction between the connected elements, which in turn is a function of the condition of faying surfaces. LRFDS Table A-J3.2 gives values of F_v for A325-SC and A490-SC bolts for various types of holes (STD, OVS, SST, LST) and Class A faying surfaces (slip coefficient 0.33; uncoated clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel). In this text, unless otherwise given, Class A surface preparation will be assumed. For this surface condition and STD holes F_v = 17 ksi for A325-SC bolts and F_v = 21 ksi for A490-SC bolts. For example, the design slip resistance for use at the service-load level for a A325-SC bolt in single shear, in a standard hole, and Class A surface preparation is given by Eq. W6.5.3a as : 1.00(17) A_b(1) = 17A_b kips.

The design slip resistance B_{dss} must equal or exceed the shear force on the bolt under service loads; that is:

$$B_{dss} \geq B_{vs} \tag{W6.5.3b}$$

When the loading combination includes wind loads in addition to dead and live loads, the total shear on the bolt due to combined load effects, at service-load level, may be multiplied by 0.75 (LRFDS Section A-J3.8*b*).

Note that the two design approaches for slip critical joints are based upon two different design philosophies:

- In the factored-load approach, the nominal slip resistance $B_{ns\,f}$ is calculated as a function of the mean slip coefficient μ and the clamping force, T_b. A factor of 1.13 (see Eq. W6.5.1) is used to account for the expected 13 percent higher mean value of the installed bolt pretension provided by the calibrated wrench method compared to the specified minimum bolt pretension T_b, used in the calculation.
- In the service load approach a probability of slip concept is used that implies a 90 percent reliability that slip will not occur at the calculated slip load B_{nss} if the calibrated wrench pretensioning method is used, or a 95 percent reliability that slip will not occur at the calculated slip load if the turn-of-nut pretensioning method is used.

The number of bolts required will be essentially the same for the two design methods because they have been calibrated to give similar results. Any small differences that occur are due to variation in live to dead load ratio (see Example W6.5.2). The factored-load level approach is provided for the convenience of only working with factored loads. The student designer should note that, irrespective of the approach, the limit state of slip is based upon the prevention of slip at the service-load levels.

The consequences of slip into bearing vary from application to application; hence the determination of which connections shall be designed and installed as slip-critical is left to judgement of the Engineer of Record. Also, the determination of whether the potential slippage of a joint is critical at the service load level as a serviceability consideration, or whether slippage could result in distortions of the frame such that the ability of the frame to resist the factored loads would be reduced can be determined only by the Engineer of Record. LRFDC Section J3.8 provides the necessary guidance.

Slip-critical joints initially fail by slipping into bearing, and then at higher load levels they fail by reaching one of the limit states considered earlier for bearing-type joints. Therefore, LRFDS Section J3.1 stipulates that a joint designed as slip-critical, in which the direction of loading is toward the edge of a connected part, must also be checked under factored loads for material bearing, using applicable provisions of LRFDS Section J3.10. Only for relatively thin plates would the number of bolts have to be increased for bearing, however.

W6.5.3 Design Tables for Slip-Critical Joints

Values of specified minimum pretension loads, T_b, for bolts in pretensioned and slip-critical joints are given in LRFDS Table J3.1 for various bolt diameters, for both A325 and A490 bolts (reproduced partially in Table 6.7.1).

The design resistance to shear at service loads, of a bolt in a slip-critical joint, for design at factored-load level (i.e. values of B_{dsf}) can be calculated using Eq. W6.5.1*a*. Values of B_{dsf} are given in the LRFDM Table 7-15 for Class A faying surfaces ($\mu =$ 0.33). The design resistance to shear at service loads, of a bolt in a slip-critical joint, for design at service-load level (i.e., values of $B_{ds\,s}$) can be calculated using Eq. W6.5.3*a*. LRFDM Table 7-16 gives values of $B_{ds\,s}$ (in kips) for Class A faying surfaces. In both these tables, values are given for A325-SC and A490-SC bolts for nominal diameters of ⅝-in. to 1½-in. in single shear and double shear. Values are given for hole types STD, OVS, SSL, LSLP and LSLT.

Table 6.7.1 is a reduced version of LRFDM Tables 7-10, 7-14, 7-15 and 7-16 for ⅝-, ¾-, ⅞- and 1-in.-dia. high-strength bolts.

W6.5.4 Numerical Examples

EXAMPLE W6.5.1 Shear Resistance of a Slip-Critical Joint

Determine the maximum axial tensile load P (30 percent dead load and 70 percent live load) that can be transmitted by the bolts in the butt splice shown in Fig. X6.13.2. The main plates are ½ in. thick and the cover plates are ⅜ in. thick. The plates are of A514 Gr 100 steel. Assume 1-in. dia. A490 bolts in standard holes with threads eXcluded from the shear planes, and Class A surface preparation. L_e = 1 ¾ in. and p = 3½ in. Consider the joint as

(a) slip-critical joint designed at the factored-load level

(b) slip-critical joint designed at the service-load level.

Solution

Bolt material: A490

From Table 2-1of the LRFDM: $F_{ub} = 150$ ksi

Bolt diameter, $d = 1.0$ in.

There are six bolts in the butt joint. So, $N = 6$

Bolt area, $A_b = \frac{\pi}{4}(1.00)^2 = 0.785$ in.^{2}

Bolts in a butt joint are in double shear. So, $N_s = 2$

Assuming sheared edges, the minimum edge distance, $L_{e\,min}$, for a 1-in. dia. bolt may be obtained from LRFDS Table J3.4 as 1.75 in.

Edge distance provided, $L_e = 1.75$ in. O.K.

Recommended minimum spacing of bolts (from LRFDS Section J3.3) = $3d = 3.0$ in.

Spacing provided, $p = 3.5$ in. O.K.

a. Slip-critical joint designed at the factored-load level

From Eq. W6.5.1a (or, from Eq. J3-1of the LRFDS), the design resistance to shear of a bolt in a slip-critical joint, for design at factored-load level, is:

$$B_{dsf} = \phi_s(1.13)\mu\, T_b N_s$$

where ϕ_s = resistance factor = 1.0 for bolts in standard holes

μ = mean slip coefficient = 0.33 for Class A faying surfaces

T_b = specified minimum bolt pretension

= 64.0 kips for a 1-in. dia. A490 bolt (from LRFDS Table J3.1, or from Table 6.7.1)

resulting in

$$B_{dsf} = 1.0\,(1.13)\,(\,0.33\,)\,(64.0)\,(2.0) = 47.7 \text{ kips}$$

Alternatively, the design resistance to shear of a bolt in a slip-critical joint, for design at factored-load level, may be obtained from LRFDM Table 7-15 (or, from Table 6.7.1). For a 1-in. dia. A490 bolt in standard holes in double shear and Class A surface preparation, the value is 47.7 kips.

The design resistance to shear of the connectors, verified at the factored load-level,

$$C_{dsf} = N B_{dsf} = 6\,(47.7) = 286 \text{ kips}$$

If P_s is the service load the six bolts are capable of transferring, then

$$1.2(0.30\,P_s) + 1.6(0.70\,P_s) \le C_{dsf} = 286$$

$$\rightarrow \quad P_s \le 193 \text{ kips}$$

Hence, $P_{s\,\max\,1} = 193$ kips (Ans.)

b. Slip-critical joint designed at the service-load level

From Eq. W6.5.3a (or, from LRFDS Section A-J3.8), the design resistance to shear of a bolt in a slip-critical joint, for design at the service-load level, is:

$$B_{dss} = \phi_{ss} F_v A_b N_s = 1.0\,(21.0)\,(0.785\,)\,(2) = 33.0 \text{ kips}$$

Alternatively, the design resistance to shear of a bolt in a slip-critical joint, for design at service-load level, may be obtained from LRFDM Table 7-16 (or, from Table 6.7.1). For a 1-in. dia. A490 bolt in standard holes in double shear and Class A surface preparation, the value is 33.0 kips.

The design slip resistance of the connectors, verified at the service load-level,

$$C_{dss} = N B_{dss} = 6\,(33.0) = 198 \text{ kips}$$

If P_s is the service load the six bolts are capable of transferring from the main plate to cover plates, we have:

$$1.0(0.30 P_s) + 1.0(0.70 P_s) \le C_{dss} = 198$$

$$\rightarrow \quad P_s \le 198 \text{ kips}$$

Hence, $P_{s\,\max 2} = 198$ kips (Ans.)

EXAMPLE W6.5.2 Design of a Slip-Critical Joint

A lap joint connecting two ½ in. plates transmits axial service tensile loads $P_D = 60$ kips and $P_L = 60$ kips using 1-in. dia. A325 high-strength bolts in standard holes with threads iNcluded in the shear plane (see Fig. X6.13.3). Assume A572 Gr 50 steel and $L_c > 2d$ for all bolts. Determine the number of bolts required for:

(a) slip-critical joint designed at the factored-load level, and

(b) slip-critical joint designed at the service-load level.

Assume Class A surface preparation.

Solution

Service load, $P_s = 1.0P_D + 1.0P_L = 1.0\times60 + 1.0\times60 = 120$ kips

Factored load, $P_u = 1.2P_D + 1.6P_L = 1.2\times60 + 1.6\times60 = 168$ kips

Diameter of bolt, $d = 1.0$ in.

Cross sectional area, $A_b = 0.785$ in.^{2}

Type of bolt: A325-SC. So, $F_{ub} = 120$ ksi

In a lap joint the bolts are in single shear $\rightarrow N_s = 1$

a. Slip-critical joint designed at the factored-load level

The design resistance to shear of a bolt in a slip-critical joint, for use at the factored-load level (from Eq. W6.5.1*a*),

$$\begin{aligned} B_{dsf} &= \phi_s(1.13)\mu T_b N_s \\ &= 1.0\,(1.13)\,(0.33)\,(51.0)\,(1) = 19.0 \text{ kips} \end{aligned}$$

Alternatively, the design resistance to shear of a bolt in a slip-critical joint, for use at the factored-load level can be read from LRFDM Table 7-15 (or, from Table 6.7.1). Corresponding to a 1-in. dia. A325-SC bolt, in a slip-critical joint with Class A faying surfaces, we obtain a value of 19.0 kips.

$$\text{Number of bolts required} = \frac{P_u}{B_{dsf}} = \frac{168}{19.0} = 8.85$$

Provide 9 bolts, or if an even number of bolts is desired, 10 bolts. (Ans.)

b. Slip-critical joint designed at the service-load level

The design resistance to shear of a bolt in a slip-critical joint, for use at the service-load level (from Eq. W6.5.3a),

$$B_{dss} = \phi_{ss} F_v A_b N_s = 1.0(17.0)(0.785)(1) = 13.4 \text{ kips}$$

Alternatively, the design resistance to shear of a bolt in a slip-critical joint, for use at the service-load level can be read from LRFDM Table 7-16 (or, from Table 6.7.1). Corresponding to a 1-in. dia. A325-SC bolt, in a slip-critical joint with Class A faying surfaces, we obtain a value of 13.4 kips.

$$\text{Number of bolts required} = \frac{P_s}{B_{dss}} = \frac{120}{13.4} = 8.96$$

Provide 9 bolts, or if an even number of bolts is desired, 10 bolts. (Ans.)

Remarks

1. Design of a joint as slip-critical requires considerably more bolts, compared to a bearing-type joint (six required in Example 6.13.3 versus nine required in this Example).

2. For the joint considered as slip-critical, both the design at the factored-load level and the design at the service-load level require the same number of bolts (nine).

3. By repeating the calculations for P_D = 30 kips, P_L = 90 kips and then for P_D = 90 kips, P_L = 30 kips, we observe that the number of bolts required for the three cases (bearing-type, slip-critical designed at factored loads, and slip-critical designed at service loads) changes from [5.94, 8.85, 8.96] to [6.36, 9.47, 8.95] and [5.51, 8.21, 8.95] respectively. Note also that the total service load is kept constant at 120 kips in all the three variants. Note that, when the *L/D* ratio is high, design of slip-critical joint at the factored-load level requires slightly higher number of bolts, compared to its design as a slip-critical joint at the service-load level.

W6.6 Behavior of a Pre-compressed Bolt-Plate Assembly in Direct Tension (Pretensioned and Slip-Critical Joints)

As mentioned earlier, ASTM A490 bolts subjected to tension under external loads (such as the ones found in hanger connections (Fig. 6.1.2*d*), diagonal bracing connections (Fig. 6.1.2*e*), beam-to-column connections, among others) are required by LRFDS to be fully pretensioned. It might seem that any external tensile load on such a high-strength bolted joint would be directly additive to the initial bolt force and would soon cause failure. This is not true, however, as shown by the simple model below [Bickford,1981; Kulak, et al., 1987].

Figure W6.6.1: Single-bolt precompressed joint under tension load.

Consider two connected elements (plates) of total thickness L and having an area of contact A_c, connected by a single high-strength bolt. The cross-sectional area of the bolt is A_b. Initially, the two plates are in contact, the bolt has no pretension (snug-tight condition) and there is no external load on the system (Fig. W6.6.1*a*). As the nut is tightened from the snug position, the bolt will elongate while the plates are compressed. Assuming that the bolt and plates remain elastic over the entire range considered, the force in each is proportional to its change of length or:

$$B = K_b \, \delta_b; \qquad C = - K_c \, \delta_c \qquad \text{(W6.6.1)}$$

where B is the force in the bolt, C the summation of contact forces between the plates, and K_b and K_c are the axial stiffness of the bolt and of the plates, respectively. δ_b represents the bolt elongation and δ_c the connected plate compression (considered negative). The negative sign in Eq. W6.6.1*b* takes care of the fact that a decrease in plate thickness (δ_c negative) results in a positive compressive force. Approximate values for K_b and K_c can be obtained from the relations

$$K_b = \frac{A_b E}{L}; \qquad K_c = \frac{A_c E}{L} \qquad \text{(W6.6.2)}$$

Since $A_c >> A_b$ for most practical cases, K_c will be much greater than K_b. Elastic curves for the bolt and plates are drawn in Fig. W6.6.2 by plotting the absolute value of the force in each on the vertical axis and the deformation of each (elongation of the bolt and compression of the plates) on the horizontal axes. Thus Equations W6.6.1*a* and W6.6.1*b* are graphically represented in Fig. W6.6.2*a* as two straight lines O_bb and O_cc respectively.

Figure W6.6.2: Force-elongation relationships of a pretensioned joint in tension.

Figure W6.6.1*c* shows a sketch of the connection at the end of the initial tightening process. As there is no external load on the system at this stage, the pretension in the bolt, B_o, must equal the resultant of the compressive contact stresses between the plates, C_o. Thus, we have

$$B_o = C_o \tag{W6.6.3}$$

Let δ_{bo} be the initial elongation of the bolt and δ_{co} the initial contraction of the plates. From Eqs. W6.6.1 and W6.6.3 we can write

$$\delta_{co} = -\frac{K_b}{K_c}\,\delta_{bo} \tag{W6.6.4}$$

Turning down the nut accommodates the difference between the deformations of the bolt and the plates. The two elastic curves (for the bolt and for the plates) can be combined into a single diagram, called the joint diagram, as shown in Fig. W6.6.2*b*.

An external, axial tensile load P is now applied to the assembly making sure that there is no bending of the plates (Fig. W6.6.1*d*). The bolt will elongate and the precompressed plates will relax and tend to expand to their original thickness. If this expansion of the plates does not exceed the initial contraction of the plates, δ_{co}, some contact pressure will still remain.

In this Phase I loading, let B_I and C_I be the tension in the bolt and the resultant of the plate compressive stresses, respectively, under the load P (Figs. W6.6.1*e* and W6.6.2*c*). Equilibrium of forces results in

$$B_I = P + C_I \tag{W6.6.5}$$

where the subscript, I, refers to the conditions in Phase I, where contact stresses between plates still remain. From Eqs. W6.6.3 and W6.6.5, we can write

$$(B_I - B_o) = P - (C_o - C_I) \tag{W6.6.6}$$

The force P acting on the system elongates the bolt – between the underside of the bolt head and the underside of nut – by an amount δ_{ba}, where

$$\delta_{ba} = \frac{B_I - B_o}{K_b} = \frac{\Delta B}{K_b} \tag{W6.6.7}$$

At the same time the thickness of the plates increases an amount δ_{ca}, due to the decrease in compression:

$$\delta_{ca} = \frac{C_o - C_I}{K_c} = \frac{\Delta C}{K_c} \tag{W6.6.8}$$

As contact is maintained, compatibility of deformations requires that:

$$\delta_{ba} = \delta_{ca} \tag{W6.6.9}$$

or

$$\frac{(B_I - B_o)}{K_b} = \frac{(C_o - C_I)}{K_c} \tag{W6.6.10}$$

These results are also shown in the joint diagram by the points b_I and c_I (Fig. W6.6.2*c*). Thus we observe that the external load increases the tension in the bolt and partially relieves the plates. The partially relieved plate partially returns to its original thickness, moving down back its elastic curve from point c_o to c_I. Simultaneously, the bolt gets longer – following its elastic curve to point b_I from b_o. We observe that the increase in length in the bolt is equal to the increase in thickness (reduction in shortening) in the connection. The connection expands to follow the nut as the bolt lengthens. Note that upper case letters are used in the equations to represent forces in the bolts and plates, while the corresponding lower case letters are used to represent the corresponding points in Fig. W6.6.2.

Substituting the value of $(C_o - C_I)$ from Eq. W6.6.10, in Eq. W6.6.6, we obtain

$$B_I - B_o = P - (B_I - B_o)\frac{K_c}{K_b} \rightarrow B_I = B_o + \frac{1}{1 + K_c/K_b}P \tag{W6.6.11}$$

The elongations δ_{ba}, δ_{ca}, the change in force in the bolt and the plates, and the final force in each are shown in Fig. W6.6.2*c*. Because of the greater stiffness of the plates, the application of the external force P results in a greater change in the plate compression, ΔC, than in the bolt tension, ΔB. That is, although the bolt tension increases, the increase is but a small fraction of the external load because of the compensating effect of the reduction in the plate compression.

A further increase in external load results in a further decrease in the plate contact pressure until the plates separate at a load P^* (Fig. W6.6.2*d*). This occurs when δ_{ca} becomes equal and opposite to the initial contraction δ_{co} of the plates. Letting $C_{\mathrm{I}} = 0$ and noting $C_o = B_o$, we obtain:

$$\frac{B^* - B_o}{K_b} = \frac{B_o}{K_c} \rightarrow B^* = B_o\left[1 + \frac{K_b}{K_c}\right] \tag{W6.6.12}$$

where B^* is the tension in the bolt when the separation just occurs. Also, from Eq. W6.6.5 with $C_{\mathrm{I}} = 0$, we have with the help of Eq. W6.6.12:

$$P^* = B_o\left[1 + \frac{K_b}{K_c}\right] \tag{W6.6.13}$$

The load P^* is known as the ***critical external load***. This is shown by the joint diagram given in Fig. W6.6.2*d*. For any P greater than P^* the plates are separated, the system becomes statically determinate, and the bolt force equals the applied load. Thus, in this Phase II of joint behavior, we have

$$B_{\mathrm{II}} = P \tag{W6.6.14}$$

and the preload has no bearing on bolt performance in this phase, as shown by the joint diagram given in Fig. W6.6.2*e*.

Figure W6.6.3: Bolt force and plate force vs. applied load.

The behavior of the bolt plate assembly is shown in a different format in Fig. W6.6.3. Here the tension in the bolt and the compression in the connection plates are shown on different sides of the horizontal axis representing the external load on the joint. Before the external load is applied we have equal and opposite preloads in the bolt and the plates. As the external load is applied, in this Phase I, the forces change in the bolt and in the plates; but the plates, being eight to twenty times stiffer than the bolt, undergo more change in force for a given change in deformation. If we apply a critical external load to the assembly, the clamping force reduces to zero. If the load is increased further, there would be a change in the slope of the bolt curve beyond this point. Now, the bolt is the only member of the assembly resisting the total external load, as the joint precompression is fully released. Thus, in this Phase II, the bolt force equals the applied load P. This relation is represented by Eq. W6.6.14 and the straight line inclined at 45° shown in Fig. W6.6.3.

The factor K_c/K_b depends on the dimensions of the connection, and for most practical cases varies between 8 and 20. Now, for elastic conditions and bolts and plates of the same material $\frac{K_c}{K_b}$ equals $\frac{A_c}{A_b}$. In an actual connection the area of plate that is compressed, A_c, is not clearly defined. If it is assumed that A_c is a circular zone whose diameter is three bolt diameters, then B^* equals $1{\cdot}11B_0$. Hence, the maximum increase in bolt force due to an applied external load, before separation takes place, is only of the order of 5 to 10 percent of the initial bolt preload. That is, until the applied load exceeds the bolt preload, the increase in bolt tension will be rather small, only about 5 to 10 percent of the applied tensile load.

In design calculations, bolt forces are generally calculated without any regard to pretension load, as though caused solely by the externally applied loads. Also, design bolt strengths in tension and in combined shear and tension are prescribed in the LRFD Specification in anticipation of this analysis procedure being followed. The true bolt force will, of course, be different from the value computed.

W6.7 Single Bolt in Shear and Tension (Slip-Critical Joint)

As seen in Section W6.6, when a bolt is pretensioned, it compresses a zone of connected plies adjacent of the bolt, so the nut travels down the shank by a distance equal to the elongation of the grip length of the bolt, plus the contraction of the connected plies. To fully overcome the clamping effect between the plies it is therefore necessary to apply sufficient external tension (equal to T^*, a force greater than the initial preload, $B_0 = T_b$). However, when an external force $B_t < T^*$ is applied to the joint the clamping force between the plates reduces from C_0 to C_1 as shown in the joint diagram of Fig. W6.6.2c. From similar triangles, and noting that P and P^* in Fig. W6.6.2 are now designated B_t and T^*, respectively, we can write:

$$\frac{C_o - C_I}{C_o} = \frac{B_t}{T^*} \rightarrow C_I = C_o\left[1 - \frac{B_t}{T^*}\right] \qquad \text{(W6.7.1)}$$

As the shear strength of a slip-critical joint is proportional to the clamping force that exists between the plates, the reduced slip resistance of the bolt may be written as:

$$B'_{nsf} = B_{nsf}\left[1 - \frac{B_t}{T^*}\right] \qquad \text{(W6.7.2)}$$

where, B_{nsf} is the slip resistance for use at factored loads, and B'_{nsf} is the slip resistance for a bolt simultaneously subject to an applied tension B_t, for use at factored loads.

In the case of slip-critical joints, the design slip resistance is proportional to the clamping force in the bolt. If external tensile loads are applied on a slip-critical joint, the clamping force is reduced below its initial value. As mentioned earlier, LRFDS permits the design of bolts in slip-critical joints to be done either at factored-load level in accordance with Sections J3.8*a* and J3.9*a*, or at service-load level in accordance with Sections J3.8*b* and J3.9*b*. In both approaches, the interaction of shear and tension is treated as linear.

W6.7.1 Design Check at the Factored-Load Level

When the design check for slip resistance is made at the factored-load level, the reduced design slip resistance of a bolt subjected simultaneously to a shear force, B_{vu}, and to a tensile force B_{tu} due to factored loads, is given by (from LRFDS Section J3.9*a*):

$$B'_{dsf} = B_{dsf}\left[1 - \frac{B_{tu}}{1.13\,T_b}\right] \quad \text{for} \quad B_{tu} \leq B_{dt} \qquad \text{(W6.7.3}a\text{)}$$

and the design criteria for the bolt becomes:

$$B_{vu} \leq B'_{dsf} \qquad \text{(W6.7.3}b\text{)}$$

where B'_{dsf} = reduced design slip resistance of a bolt simultaneously subjected to a tensile force B_{tu} under factored loads

B_{dsf} = design slip resistance of the bolt subject to shear only, according to LRFDS Section J3.8*a* (Eq. W6.5.1*a*)

T_b = specified minimum bolt pretension (Eq. 6.5.1 or LRFDS Table J3.1)

B_{tu} = tensile force in the bolt at the factored-load level (tensile component of applied factored load on the bolt for combined shear and tension loading)

B_{vu} = shear force in the bolt at the factored-load level

The factor 1.13 in the denominator accounts for the expected 13 percent higher mean value of the installed bolt pretension provided by the calibrated wrench pretensioning method compared to the specified minimum bolt pretension, T_b, used in the calculation [Carter et al., 1997]. In the absence of other field test data, this value is used for all methods of bolt pretensioning.

W6.7.2 Design Check at the Service-Load Level

When the design check for slip resistance is made at the service-load level, the reduced design slip resistance of a bolt simultaneously subjected to a shear force, B_{vs}, and to a tensile force, B_{ts}, due to service loads, is given by the relation (see LRFDS Eq. A-J3.1):

$$B'_{dss} = B_{dss}\left[1 - \frac{B_{ts}}{0.8\,T_b}\right] \qquad \text{(W6.7.4}a\text{)}$$

and the design criteria for the bolt becomes:

$$B_{vs} \leq B'_{dss} \qquad \text{(W6.7.4}b\text{)}$$

where B'_{dss} = reduced design slip resistance of a bolt subjected simultaneously to a tensile force B_{ts} under service loads

T_b = specified minimum bolt pretension from Table J3.1 of the LRFDS

B_{ts} = tensile force in the bolt at the service-load level (tensile component of applied service load on the bolt for combined shear and tension loading)

B_{vs} = shear force in the bolt at the service-load level

B_{dss} = design slip resistance of the bolt subject to shear only , according to LRFDS Section J3.8*b* (Eq. W6.5.3*a*)

The factor 0.8 represents a slip probability factor that reflects the distribution of actual slip coefficient values about the mean, the ratio of mean installed bolt pretension to the specified minimum bolt pretension (T_b), and a slip probability level [Section 5.4.2, RCSC, 2000]. The above relation is shown in Fig. W6.7.1 in a stress format.

Figure W6.7.1: Combined tension and shear of bolts in slip-critical joints (at the service-load level).

W6.8 Load Distribution in Axially Loaded Bolted Joints (Bearing-Type Joint)

In the conventional design of axially loaded bolted joints all bolts are assumed to be equally loaded. In what follows, we will study the validity of this assumption, using a simplified model, known as the elastic-plate model [Hrennikoff, 1934; Kulak et al., 1987].

Figure W6.8.1: Force distribution in an axially loaded bolted joint.

A tensile force T is transmitted from a plate A to a plate B by a bearing-type joint, using three bolts at equal spacing, p. The three bolts are of the same size and material, and the two plates are also identical. The load is applied through the centroid of the bolt group in the plan view as shown in Fig. W6.8.1*a*. The following simplifying assumptions will be made:

- There is no misalignment of holes.
- There is no clearance between the hole and the bolt, i.e., the bolt and the hole have the same diameter.
- There is no friction between the faying surfaces.
- Force transfer between plates takes place at the center line of each bolt.
- The small eccentricity e (= t) between the forces T (Fig. W6.8.1*b*) is neglected.
- The stress in each plate is uniform between bolts.
- The plates and bolts are elastic.

Owing to symmetry, the outer bolts will carry the same force. Thus, if B_1 is the force in bolt 1, we have

$$B_3 = B_1; \qquad B_2 = (T - 2B_1) \tag{W6.8.1}$$

As we have three unknowns and two relations, the problem is indeterminate to the first degree. The distribution of forces in the plates and bolts is as shown on the free body diagrams of Fig. W6.8.1*c*. The tensile force in each plate between the bolts 1 and 2 differs from that between bolts 2 and 3, as shown in Fig. W6.8.1*c*. Each plate segment will elongate from the original length (= bolt pitch, p), to the values shown in Fig. W6.8.1*d*. The outer bolt will deform an amount Δ_1 and the inner bolt by an amount Δ_2. A compatibility relation for deformations could now be written with the help of Fig. W6.8.1*d* as:

$$\Delta_1 + \left[p + \frac{B_1 p}{AE} \right] = \Delta_2 + \left[p + \frac{(T - B_1)}{AE} p \right]$$

or

$$\Delta_2 = \Delta_1 - \frac{(T - 2B_1)}{AE} p \qquad \text{(W6.8.2)}$$

Here A is the cross sectional area of the plate and E is the Young's modulus of the plate material. Assuming that the deformation of each bolt is a linear function of the force acting upon it, the deformations of the bolts may be written as:

$$\Delta_1 = \frac{B_1}{K_b}; \quad \Delta_2 = \frac{(T - 2B_1)}{K_b} \qquad \text{(W6.8.3)}$$

where K_b is the shear stiffness (force per unit deformation) of the bolt. K_b is a function of the bolt size, length, material properties and of the manner in which the force is transmitted from the plate to the bolt. Substituting the above expressions for Δ_1 and Δ_2 in Eq. W6.8.2 and then solving for B_1, the force in the outer bolt is obtained as:

$$B_1 = \frac{[K_p + K_b]}{[3K_p + 2K_b]} T \qquad \text{(W6.8.4)}$$

Here, K_p is the axial stiffness of the plate (= EA/p). Substitution of the value of B_1 from Eq. W6.8.4 in Eq. W6.8.1 gives the force in the inner bolt as:

$$B_2 = \frac{K_p}{[3K_p + 2K_b]} T \qquad \text{(W6.8.5)}$$

When the bolts are very stiff compared to the plate (K_p is small compared to K_b or $K_p/K_b \to 0$), we obtain $B_1 = B_3 \to T/2$ and $B_2 \to 0$. That is, practically all the force will be transmitted by the outer bolts, leaving the center bolt almost unstressed. On the other hand, when the plates are absolutely rigid (K_b is very small compared to K_p or $K_b/K_p \to 0$), we obtain $B_1 = B_2 = B_3 = T/3$. That is, the applied load is divided equally among the three bolts, and they will be stressed equally. This is often referred to as the ***rigid plate theory distribution*** because it is the theoretical result obtained when the plate is very rigid relative to the bolts ($\Delta_1 = \Delta_2$). This is the only case in the elastic domain, when the applied load is distributed equally between all the bolts. In all other cases, the end bolts carry larger shares of the applied load than the middle bolt. For example, when $K_p = K_b$, $B_1 = B_3 = 0.4T$ and $B_2 = 0.2T$. Of course, this means that, as the applied load is increased gradually, the end bolts reach the yield limit before the middle bolt does. After this stage the lightly loaded middle bolt picks up a larger share of the additional load. If the bolt material is sufficiently ductile and the connection statically loaded, the end bolts which reach their ultimate strength first, may hold together until the middle bolt has also been stressed close to the ultimate strength. Thus, at failure of the joint the bolt forces are either equal or nearly so. However, the extent to which this redistribution of load (from the heavily loaded outer bolts to the lightly loaded inner bolt) can develop depends on the ability of the bolts to undergo large shear deformations.

The same procedure can be used to determine the elastic force transfer in joints having a greater number of bolts. Each additional transverse row of bolts represents another redundant element (say the force in a bolt in that row). By writing the same number of compatibility equations, we obtain a set of simultaneous equations, whose solution gives the unknown forces acting on each bolt. The results of such analyses show that, the greater the number of bolts in a line, the greater is the difference between the loads carried by the end bolts and center bolts.

Several general remarks can be made from these qualitative studies.

- As long as a connection remains elastic, the distribution of bolt forces is not uniform except in the extreme case in which the plates are very rigid compared to the bolts.
- The longer the joint (i.e., the greater the number of bolts parallel to the line of force T), the greater will be the proportion of load transmitted by the outer bolts. Again, this is valid only as long as the connection remains elastic.
- If yielding on the gross area of the plates can be postponed until after there has been some nonlinear behavior of the bolts, a better redistribution of the unequal bolt forces will result. On the other hand, early yielding of the plates accentuates the inequalities in bolt forces and causes premature failure of those at the ends of the joints. For this reason, A325 bolts joining plates of a high strength steel are more efficient than the A325 bolts joining plates of a low yield steel. Similarly, A490 bolts joining A36 steel plates are less efficient than A325 bolts joining A36 steel, given the same geometry.

The study, although approximate, indicates that it is desirable to arrange a joint compactly in order to equalize the loads on the bolts as much as possible.

In a connection having several bolts in line, parallel to the applied load, when slip occurs, only the end bolts come into bearing against the main plate and the cover plates. As the load increases, these bolts deform until the next interior bolts are bearing. The process continues until all the bolts bear against the plates. This does not mean that each bolt is carrying an equal share of the total load at this stage [Fisher, 1965]. As the load is increased, the plate deforms plastically and finally an end bolt fails because it is over stressed. Tests indicate that in short connections with few bolts per line, almost complete equalization of load occurs before the end bolts fail [Fisher et al., 1978], due to ductility of bolt and plate material. However, each bolt may deform inelastically a somewhat different amount before the ultimate load is attained. In longer connections, on the other hand, the end bolts may reach a critical shear deformation and fail before the full strength of all bolts can be attained. This premature, sequential failure of bolts, progressing inward from the ends of the joint, is called ***unbuttoning***. The phenomenon has been observed in tests of long riveted and high-strength bolted joints [Bendigo et al., 1963]. To account for this failure mode, for bolts in joints longer than 50 inches in length, the design shear strength given by Eq. 6.7.3 is to be multiplied by an additional reduction factor of 0.80. Tests by Bendigo et al. [1963] have also indicated that the ultimate shear strength of high-strength bolts having a grip of eight or nine diameters is no less than that of similar bolts with much shorter grips.

W6.9 Additional Considerations for Fillet Welds

W6.9.1 Fillet Weld Terminations and End Returns

Figure W6.9.1: Fillet weld terminations.

Fillet welds are permitted to extend to the ends or sides of parts, or be stopped short of ends, or boxed, with the following exceptions:

- At lapped joints where one part extends beyond an edge subject to calculated tensile stress at the start of the overlap, these fillet welds must terminate at a distance not less than the size of the weld, *w,* from the stressed edge. As an example, for the lap joint between the web member of a truss and the stem of the WT tension chord member, the weld should not extend to the edge of the WT stem (Fig. W6.9.1). A way to avoid inadvertent notches at this critical location is to strike the welding arc at a point slightly back from the edge and proceed with welding in the direction away from the edge.
- Where framing angles extend beyond the end of the beam web to which they are welded, the free end of the beam web is subjected to zero stress; then it is permissible for the fillet weld to extend continuously at the top end, along the side, and along the bottom end of the angle to the extreme end of the beam (see Weld A in Figs. 13.4.1*b* and *d*).

Fillet welds that extend to the ends or sides of parts are sometimes continued around the corners for a distance not less than two times the nominal size of the weld (see Weld B in Figs. 13.4.1*c* and *d*). Such additional welds, called ***end returns***, are used to ensure that the weld size is maintained over the length of the weld. They are also useful in reducing the high stress concentrations which occur at the ends of welds and in increasing the plastic deformation capability of the connection. End returns of the above length have negligible effect on the static strength of the connection, but they do significantly increase the fatigue resistance of cyclically loaded flexible end connections. End returns must be indicated on the design and shop drawings. Most designers do not consider end returns, of such short lengths as mentioned above, to participate in resisting load. However, there is no LRFD provision prohibiting such consideration. The LRFD Specification makes specific reference to end returns in Section J2.2*b*.

- For connections which are subject to maximum stress at the weld termination due to cyclic forces and/or moments of sufficient magnitude and frequency to initiate a progressive fatigue failure emanating from unfilled start or stop craters or other discontinuities, the end of the weld must be protected by returns or boxing. If the bracket is a plate projecting from the face of a support, extra care must be taken in depositing the boxing weld across the thickness of the plate to assure that a weld free of notches is deposited.
- For connections (such as framing-angle-to-support-connections), which are assumed in analysis of structure as simply-supported, and which therefore, depend upon the flexibility of the outstanding elements for providing the necessary flexibility of the

connection, the top and bottom edges of the outstanding legs must be left unwelded over a substantial portion of their length in order to assure such flexibility. Tests by Johnston and Green [1940] indicated that the static strength of such connections is the same with or without end returns. The use of returns is therefore optional. The end returns, if used, must have a length not less than two times the nominal weld size but not more than four times the nominal weld size.

- The cover plate and beam seat angle connection details shown in Figs. W6.9.2*a* and *b* give two situations where the welds, lie on opposite sides of a common contact surface. Any attempt to connect the welds would tend to melt the corner material, reducing the thickness and creating a notch. The presence of a notch increases the risk of brittle fracture. Such fillet welds located on opposite sides of a common plane shall be interrupted at the corner common to both welds for a distance equal to the nominal weld size from the corner. The cover plate and the seat angle connection details referred to above could be changed to those as shown in Figs. W6.9.2*c* and *d* respectively, to permit the welds to be properly returned.

Figure W6.9.2: Situations where end returns are prohibited.

W6.9.2 Flat Bar Tension Member with Longitudinal Fillet Welds Only

Figure W6.9.3: Flat bar tension member with longitudinal fillet welds only.

As shown in Fig. W6.9.3, when longitudinal fillet welds are used alone (i.e., without an end transverse weld) in end connections of flat bar tension members, the length of each fillet weld, L_{lw}, may not be less than the width of the plate, W_p, between them. This is done to reduce shear lag effects. Thus:

$$L_{lw} \geq W_p \qquad \text{(W6.9.1)}$$

W6.9.3 Overlap of Lap Joints

Figure W6.9.4: Overlap of lap joints.

In lap joints, the minimum amount of overlap, L_{lap}, shall be five times the thickness of the thinner part joined, but not less than 1 in. (Fig. W6.9.4). That is (LRFDS Section J3.2*b*):

$$L_{\text{lap}} \geq \max(5t_{p1};\ 1\text{ in.}) \qquad \text{(W6.9.2)}$$

where L_{lap} = lap length provided

t_{p1} = thickness of the thinner plate joined

W6.9.4 Shelf Dimension for a Fillet Weld

Figure W6.9.5: Shelf dimensions for fillet welds.

Designers should detail connections to insure that welders have enough space for positioning and manipulating electrodes, and for depositing the required size of fillet weld. Stick electrodes may be up to 18 in. long and ⅜ in. in diameter. For example, to provide a SMAW fillet weld of size w, the projection or shelf dimension b should at least be $w + \frac{5}{16}$ in. (Fig. W6.9.5). SAW welds would require a greater shelf dimension to contain the flux. In building column splices, however, a smaller shelf dimension $b = w + \frac{3}{16}$ in. is often used for welding splice plates to filler plates.

W6.10 Strength of Fillet Welds

W6.10.1 Distribution of Stresses Along Fillet Welds

Figure W6.10.1: Distribution of stresses along fillet welds.

The actual stress distribution in a welded joint is complex and non-uniform. This is especially true of a longitudinal fillet weld in a lap joint (Fig. W6.10.1*a*). The stress in the narrow plate is zero at A and a maximum at B, while that in the wider plate is zero at B and a maximum at A. At some intermediate point O the stresses in the two plates are equal. Between O and B the stress increases in the narrow plate in the direction OB, while that in the wider plate is decreasing in that direction. This differential between the intensities of the stresses in the two plates is absorbed by the weld and hence the point of maximum stress in the weld is at B, with a minimum at point O. By similar reasoning, there is a high point in the weld stress at A. The actual variation of shear stress in the weld from point A to point B depends on the length of the weld as well as the ratio of the widths of the plates being joined. In a ductile weld, such as is obtained with a properly coated electrode, the more highly stressed portions of the weld yield first when the elastic limit is reached under increasing load. Thus, due to the ductility of the weld metal, redistribution of stress occurs, and the curve of Fig. W6.10.1*a* flattens out and approaches more nearly a horizontal line. The stress concentrations at the ends of longitudinal welds are of no consequence for static or quasi-static loads. But their influence on joint behavior could be significant where fatigue loads are involved. A recommended method to reduce these high stress concentrations at the ends of fillet welds, and to inhibit cracking and progressive tearing throughout their length is to hook the welds around the ends and provide ***end returns***, as discussed in Section W6.9.1. Figure W6.10.1*b* shows the typical shear stress variation in a transverse fillet weld.

W6.10.2 Load-Deformation Relationship of a Fillet Weld Element

Unlike bolts, fillet welds are continuous connectors. The maximum strength and deformation sustained by an element of fillet weld is a function of the angle θ the resultant force on the element makes with the axis of the weld element. The load-deformation relations for single unit-length weld elements were first obtained by Butler and Kulak [1971], and by Butler, Pal, and Kulak [1972], who conducted a series of 23 tests on ¼- in. fillet welds using E60 electrodes. Additional curves for welds with E70 electrodes were obtained by Kulak and Timmler [1984]. Strength curves for E70 electrodes were also developed by Lesik and Kennedy [1990] and form the basis for the relations given in LRFDS Appendix J2.4. One such set of load-deformation relations, in non-dimensional form, are shown in LRFDM Fig. 8-6 for θ= 0°, 30°, 45°, 60°, 75°, and 90°. It is observed that the transverse welds (θ = 90°) are 40 to 50 percent stronger than longitudinal welds (θ = 0°), while longitudinal welds are four times more ductile than transverse welds. Two such curves (for θ = 0°, and θ = θ°) are schematically shown in Fig. W6.10.2.

Figure W6.10.2: Load-deformation response of unit-length fillet welds.

The maximum strength of a fillet weld element, when the resultant elemental force is along the longitudinal axis of the weld element (i.e., θ= 0) is given by:

$$R_{0m} = 0.6\, F_{\mathrm{EXX}}\, A_w \qquad \text{(W6.10.1)}$$

where R_{0m} = maximum strength of a fillet weld element, when the resultant elemental force is along the longitudinal axis of the element (i.e., for θ= 0), kips

F_{EXX} = minimum specified ultimate tensile stress of the weld electrode ($\equiv F_{uw}$), ksi

A_w = effective throat area of the weld element, in.^{2}

= $t_e\, L_w$

Also, the maximum strength of a fillet weld element when the resultant elemental force makes an angle θ with the axis of the weld element is given in LRFDS Section A-J2.4:

$$R_{\theta m} = \left(0.6 F_{\mathrm{EXX}} A_w\right)\left[1.0 + 0.50(\sin\theta)^{1.5}\right] \quad \text{(W6.10.2)}$$

The deformation of the weld element at maximum strength is given by

$$\Delta_{\theta m} = 0.209(\theta + 2)^{-0.32} w \quad \text{(W6.10.3)}$$

where w = leg size of the fillet weld, in.

θ = angle of loading measured from the weld longitudinal axis

The strength of a fillet weld element when the resultant elemental force makes an angle θ with the axis of the weld element, and corresponding to a deformation δ, is given by

$$R_{\theta\delta} = R_{\theta m} f(p) \quad \text{(W6.10.4)}$$

where

$$f(p) = \left[p\left(1.9 - 0.9p\right)\right]^{0.3} \quad \text{(W6.10.5}a\text{)}$$

with

$$p = \frac{\delta}{\Delta_{\theta m}} \tag{W6.10.5b}$$

or more explicitly:

$$R_{\theta\delta} = \left(0.6 F_{EXX} A_w\right)\left[1.0 + 0.50(\sin\theta)^{1.5}\right]\left[\frac{\delta}{\Delta_{\theta m}}\left(1.9 - 0.9\frac{\delta}{\Delta_{\theta m}}\right)\right]^{0.3} \tag{W6.10.6}$$

where $R_{\theta\delta}$ = strength of a fillet weld element for given values of θ and δ, kips

F_{EXX} = minimum specified ultimate tensile stress of the weld electrode, ksi

A_w = effective area of weld element (= $t_e L_w$), in.^{2}

t_e = effective throat thickness, in.

L_w = effective length of weld element, in.

θ = angle of loading measured from the weld longitudinal axis, degrees

$\Delta_{\theta m}$ = deformation of the weld element at maximum strength for a given θ, in.

δ = deformation of the weld element, in.

$R_{\theta m}$ = maximum strength of the weld element for a given θ

The ductility or the ultimate deformation of the weld is also a function of θ. The deformation $\Delta_{\theta f}$ at fracture of a weld element, when the resultant elemental force makes an angle θ with the axis of the weld element, is given by:

$$\Delta_{\theta f} = \min[1.087(\theta + 6)^{-0.65} w;\ 0.17w] \tag{W6.10.7}$$

W6.10.3 Effective Length of an End-Loaded Fillet Weld

Figure W6.10.1 shows the typical stress distribution at service load for end-loaded fillet welds in a lap joint. Even though the elastic stress distribution along the length of weld is not uniform, almost complete equalization of stresses occurs in short connections before the end weld elements fail, due to ductility and plastic deformation of weld elements and plates. Thus, for end-loaded fillet welds with a length up to 100 times the leg dimension, LRFDS Section J2.2*b* permits the effective length of weld to be taken as the actual length. When the length of the end-loaded fillet weld exceeds 100*w*, the effective weld length must be determined by multiplying the actual length by a reduction factor, β, where:

$$\beta = \min[\{1.2 - 0.002\,(L_{lw}/w)\};\ 1.0] \quad \text{for } 100\,w < L_{lw} \le 300\,w \tag{W6.10.8a}$$

$$= 0.6 \quad \text{for } L_{lw} > 300\,w \tag{W6.10.8b}$$

where L_{lw} = actual length of end-loaded fillet weld, in.

w = weld leg size, in.

W6.11 Maximum Effective Fillet Weld Size, w^*_{BM}

Figure W6.11.1: Critical shear planes for fillet welds loaded in longitudinal shear.

The student designer should understand that "over-welding" is one of the major factors of welding cost. In what follows, we determine for welds loaded in longitudinal shear only, a ***maximum effective fillet weld size***, w^*_{BM}, above which weld strength is limited by the strength of the base metal rather than the weld. *That is, no useful purpose will be served by using a bigger size weld than* w^*_{BM}.

Consider the fillet weld shown in Fig. W6.11.1*a*. This weld is transmitting, from plate A to plate B, a shear force acting in the same direction as the axis of the weld. Fig. W6.11.1*a* illustrates the shear planes for the fillet weld and base material defining three possible limit states of failure:

- Plane 1-1, in which the resistance is governed by the shear strength for material A
- Plane 2-2, in which the resistance is governed by the shear strength of the weld metal
- Plane 3-3, in which the resistance is governed by the shear strength of the material B.

The resistance of the welded joint is the lowest of the resistances calculated for each of these three planes of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. In effect, tests have shown that the stress in this fusion area is not critical in determining the shear strength of the fillet welds [Preece, 1968].

The design strength of fillet weld, R_{dw}, along plane 2-2 is given by Eq. 6.19.4. The design shear strength of the base material, R_{dBM}, along plane 1-1 and 3-3 is given by Eq. 6.19. 6. Let us assume that the two plates are of the same thickness and material, and that the welds are made by the SMAW process. To achieve economy of the weld and not to over design the weld, we assure that the weld strength controls the design. That is:

$$R_{dw} \leq R_{dBM} \tag{W6.11.1}$$

If we consider a unit length of the connection, for the weld strength to control the design, we obtain for steels other than A514:

$$0.75(0.6F_{\mathrm{EXX}})(0.707w)(1) \leq 0.9(0.6F_{y\mathrm{BM}})t_p(1) \rightarrow w \leq w^*_{\mathrm{BM}}$$

with

$$w^*_{\mathrm{BM}} \leq 1.70 \frac{F_{y\mathrm{BM}}}{F_{\mathrm{EXX}}} t_p \tag{W6.11.2}$$

where w = fillet weld leg size, in.

w^*_{BM} = weld size above which weld strength is limited by the strength of the base material

t_p = thickness of the base material, in.

$F_{y\mathrm{BM}}$ = yield stress of the base material, ksi

F_{EXX} = ultimate tensile stress of the weld material, ksi

To illustrate, we obtain:

for $F_{yBM} = 36$ ksi, $F_{EXX} = 70$ ksi, $\qquad w^*_{BM} = 0.874\, t_p$

for $F_{yBM} = 50$ ksi, $F_{EXX} = 70$ ksi, $\qquad w^*_{BM} = 1.21\, t_p$

Note that w^*_{BM} cannot exceed w_{max} defined in Eq. 6.16.1, nor t_p.

Next consider the two fillet welds connecting the flange and web of a plate girder transferring the horizontal shear flow between the two members. The shear load is again parallel to the weld axis. Figure W6.11.1*b* illustrates the three shear planes of failure. Sections 3-3 through the flange will not be critical. Again, for the weld strength to control the design, we require that R_{dw}, the design strength of the two fillet welds per unit length (sections 2-2), be less than or equal to $R_{d\,BM}$, the design strength in shear of the web plate (section 1-1). Or:

$$0.75(0.6F_{EXX})\,2\,(0.707\,w)\,1 \le 0.9(0.6F_{yBM})\,t_p\,(1) \qquad \rightarrow \quad w \le w^*_{BM} \qquad \text{(W6.11.3)}$$

with

$$w^*_{BM} = 0.85\,\frac{F_{yBM}}{F_{EXX}}\,t_p \qquad \text{(W6.11.4)}$$

where w = leg size of each of the two fillet welds, in.

w^*_{BM} = weld size above which weld strength is limited by the strength of the base material, in.

t_p = thickness of the base material, in.

F_{yBM} = yield stress of the base material, ksi

F_{EXX} = ultimate tensile stress of the weld material, ksi

To illustrate again, we obtain:

for $F_{y\,BM}$ = 36 ksi, F_{EXX} = 70 ksi, $w^*_{BM} = 0.437\ t_p$

for $F_{y\,BM}$ = 50 ksi, F_{EXX} = 70 ksi, $w^*_{BM} = 0.607\ t_p$

If the actual weld size provided is greater than w^*_{BM} given in Eqs. W6.11.2 or W6.11.4 (to satisfy the minimum weld size requirements of LRFDS J2.4, for example), the design strength of that fillet weld should be calculated using $w = w^*_{BM}$.

Another situation in which the concept of the maximum effective weld size applies is when two longitudinal fillet welds connect a bracket plate to a column flange (Fig. 6.1.3*h*), since (for small eccentricities) the transfer of load by means of the weld is a shear transfer to the column flange.

W6.12 Groove Welds

W6.12.1 Groove Weld Definitions

Figure W6.12.1: Nomenclature of groove welds.

The nomenclature of groove welds is given in Fig. W6.12.1. The terms refer to the preparation of material and the relationship of abutting parts, as well as to the welds themselves. The ***root opening*** is the separation between the pieces being joined (Figs. W6.12.1*a*, *b*) and is provided for electrode accessibility to the base or root of the joint. The smaller the root opening the greater must be the angle of the bevel. In the case of single bevel and single V welds, a root opening of 1/16 in. must be provided. For single J and single U welds, root openings may vary from 0 to 1/8 in. The root openings for the double bevel and double V welds are 1/8 in. Those for the double J and double U may vary from 0 to 1/8 in. The root of the initial pass must be gouged out before welding from the back side is started.

A ***land*** is used to provide an additional thickness of metal, as opposed to a ***feather edge***, in order to minimize any burn-through tendency. To obtain complete fusion when welding a plate, back gouging is required on all joints except "Vees" with feather edge. This may be done by grinding, chipping or arc-air gouging (cost economical).

Backing bars or ***backup strips*** are commonly used when all welding must be done from one side, or when the root opening is excessive (Figs. W6.12.1*a*, *c*). The backing bar is often ¼ in. copper plate. Weld metal does not stick to copper and copper also has a very high conductivity which is useful in carrying away excess heat and reducing distortion.

Sometimes steel backing bars are used. They are generally left in place. Short intermittent tack welds, generally staggered, are used to connect the backing bars to the plates.

A butt weld is usually convex on one or both sides, and the added weld metal that causes the throat dimension to be greater than the thickness of the welded material is known as ***reinforcement*** (Fig. W6.12.1*d*). However, the reinforcement is ignored in evaluating the throat size. In fact, reinforcement is objectionable as it results in stress concentration and should not in any case exceed $\frac{1}{8}$ in. The ***size of a butt weld***, *w*, is taken equal to the thickness of the plates being joined if the plates are of the same thickness, or equal to the thickness of the thinner plate if the plates to be joined are of different thickness.

The type of groove selected depends upon the thickness of the material, the position of the weld, and whether only one side or both sides are accessible for welding. The most economical type of groove to select also depends upon the fabricating shop which does the work, and upon the kind of welding equipment which can be used. The square groove welds are used to connect relatively thin material up to roughly $\frac{5}{16}$ in. thickness. The root opening must not be less than one-half the thickness of the thinner plate. Furthermore, this weld must be deposited using a backing strip or welded in the flat position from both sides. Above $\frac{5}{16}$ in. in thickness, some form of beveling is desirable to insure penetration. Plate edges are beveled to provide accessibility to all parts of the joint and to insure good fusion throughout the entire weld cross section. The choice of bevel angle is essentially a compromise between a large angle for control and a small angle to minimize the volume of deposited metal. The single Vee is prepared simply by burning the edge of the plates away with a torch. For thicker plates the volume of weld metal may be reduced by changing from single Vee to single U groove welds.

Single grooves are cheaper to form but require more weld than the double groove joints. For example, the single Vee joint requires approximately twice as much weld metal as the double Vee. However, double groove welds require more labor in edge preparation and in cleaning of the weld root prior to starting the weld on the second side. So, many fabricators use single groove welds in thicknesses up to about one inch. With larger plate thicknesses the double Vee weld will require too much weld metal, and therefore a double U weld is used with slopes of 20° or smaller. Bevel or Vee grooves can usually be flame cut, and therefore are less expensive than J- and U-grooves which require planing or arc-air gouging. For horizontal welds bevel or V welds are preferable, because it is difficult to make a good J or U weld in this position.

The distortion of welded components increases with the amount of heat input and is proportional to the cross section of the fused metal. The designer can reduce distortion by using butt welds in lieu of fillet welds, double groove welds instead of the single groove welds, and by using U-grooves rather than V-grooves.

Groove welds are also classified as either complete-joint-penetration or partial-joint-penetration welds. A ***complete-joint-penetration (CJP) groove weld*** is one in which fusion of base and weld metal are achieved throughout the depth of the joint (Fig. W6.12.1*d*). ***Partial-joint-penetration (PJP) groove welds*** extend for only part of the member thickness (Fig. W6.12.1*e*). They are used when stresses to be transferred are substantially smaller than those that would require CJP groove welds, or when welding must be done from one side of a joint only and it is not possible to use backing bars or to gouge weld roots for back welds. PJP groove welds may also be made from both sides. Edge preparation of base material for PJP groove welds is similar to that for CJP groove welds, but it usually covers less than the full joint thickness. Partial-joint-penetration groove welds are generally used for welded column splices, for connecting together various elements of built-up box sections for truss chords and pedestals, among others. They are not recommended in joints subject to dynamic or cyclical loading, except for joining the components of built-up members.

W6.12.2 Effective Area of Groove Welds

The ***effective area of a groove weld*** is the product of the effective length of the weld times the effective throat thickness. The ***effective length of a groove weld*** is the width of the part joined. The ***effective throat thickness*** of a complete-joint-penetration groove weld is the thickness of the thinner plate joined. No allowance is made for the reinforcement at the weld. Thus

$$t_e = t_{p1} \tag{W6.12.1}$$

where t_{p1} = thickness of the thinner plate joined

t_e = effective throat thickness of a groove weld

W6.12.3 Design Strength of CJP Groove Welds

The design strength of welds is determined in accordance with LRFDS Sections J2, J4, and J5. Two limit states, namely, the limit state of weld-metal strength and the limit state of base-metal strength, must be checked following LRFDS Table J2.5. Design strength of the weld is given by

$$R_d = \min\left[R_{dw},\ R_{d\mathrm{BM}}\right] \qquad \text{(W6.12.2)}$$

Groove welds may be stressed in tension, compression, shear, or a combination of these, depending upon the direction and position of the load relative to the weld. The design strength of a complete-joint-penetration groove weld depends upon the type of stress that is applied.

- For tension or compression normal to effective area, or tension or compression parallel to axis of weld:

$$R_{dw} = 0.90\, F_{yw}\, t_e\, L_w; \qquad R_{d\mathrm{BM}} = 0.90\, F_{y\mathrm{BM}}\, t_e\, L_w \qquad \text{(W6.12.3)}$$

where L_w = effective length of the groove weld, in.

t_e = effective throat thickness of the groove weld, in.

F_{yw} = yield stress of the weld metal, ksi

$F_{y\,\mathrm{BM}}$ = yield stress of the base material, ksi

For the case of tension normal to the effective area, matching filler metal, as given in Table 4.1 of AWS D1.1 must be used always. Consequently, for this case, $F_{y\mathrm{BM}}$ will always control over F_{yw}. For the cases of compression normal to effective area, and tension or compression parallel to axis of weld, matching filler metal or filler metal one classification lower (10 ksi) may be used. When matching filler metal is used, $F_{y\mathrm{BM}}$ will again control over F_{yw}.

- For shear on effective area:

$$R_{dw} = 0.80\left(0.6F_{\mathrm{EXX}}\right)t_e L_w; \quad R_{d\mathrm{BM}} = 0.90\left(0.6F_{y\,\mathrm{BM}}\right)t_e L_w \qquad \text{(W6.12.4)}$$

where F_{EXX} = minimum specified tensile strength of filler metal

$F_{y\,\mathrm{BM}}$ = yield stress of the base material

We observe that the shear yield stress of the filler metal is approximated as 0.6 of the ultimate tensile stress of the filler metal to calculate the filler metal strength, and the shear yield stress of the base material as 0.6 of the yield stress of the base material to calculate the base material strength. Consequently a lower ϕ factor of 0.80 is used for the weld metal design strength.

For the case of shear on effective area, matching filler metal or filler metal one classification lower (10 ksi) may be used. When matching electrode material is used, the base material will again control the design strength of the weld.

It is recommended that matching filler metal always be used for all complete-joint-penetration groove welds. When these filler metals are used, welds are as strong as, if not stronger than the original members being joined. Thus, when matching filler metals are used, CJP groove welds do not really have to be designed.

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Figure W6.1.1 **Direct tension indicator** TO COME SHORTLY.

Figure W6.1.2 **Twist-off-type tension control bolt** TO COME SHORTLY.

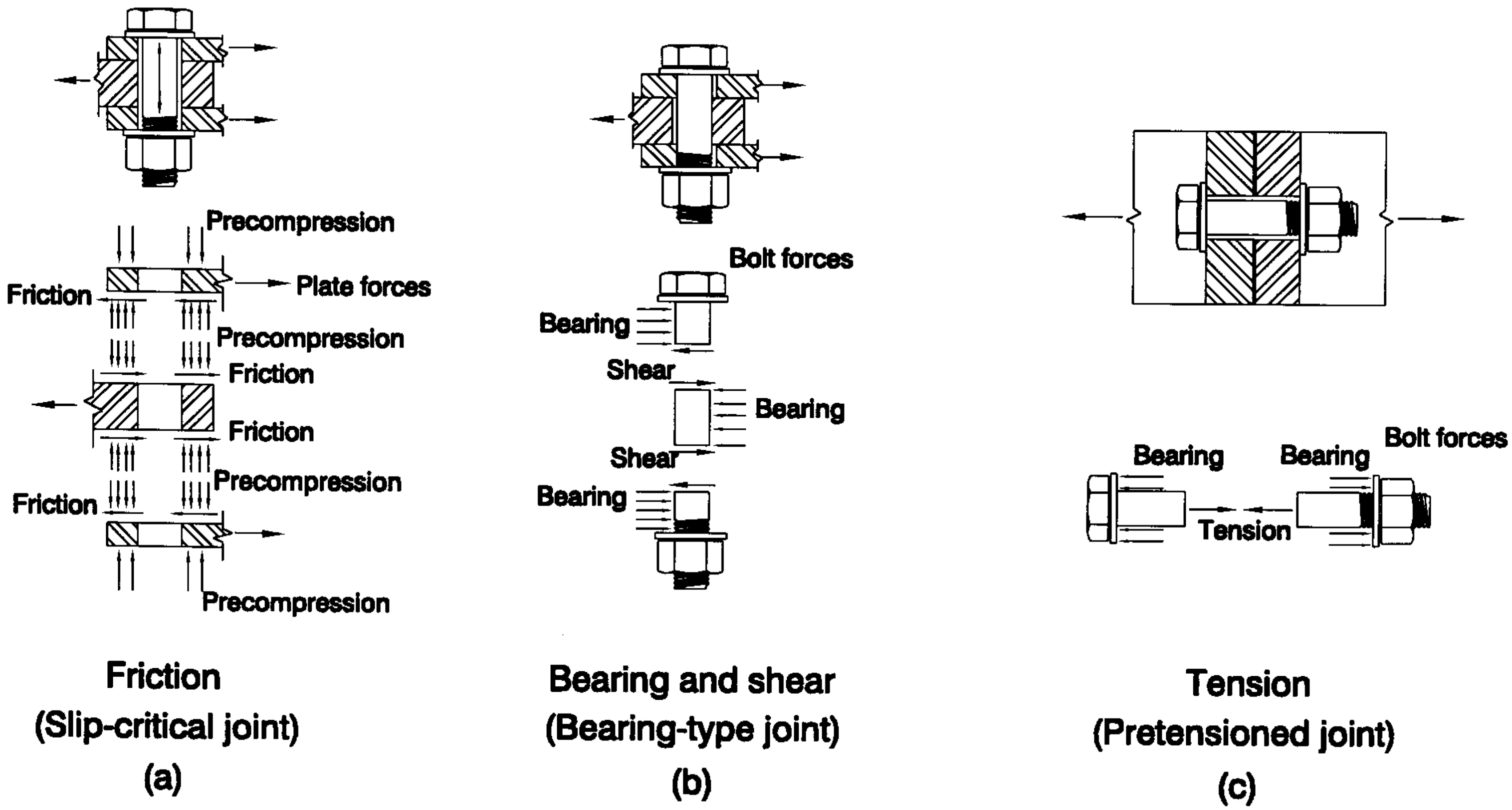


Figure W6.2.1: Force transfer mechanisms in bolted joints.

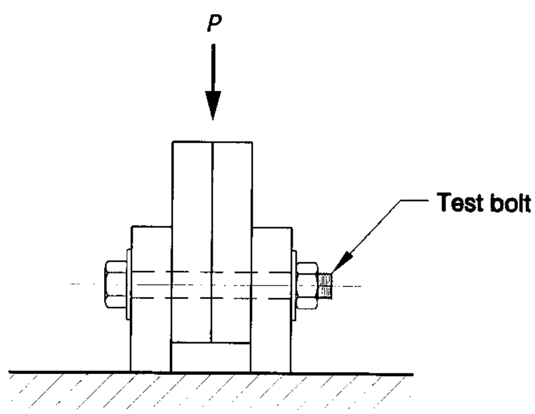


Figure W6.3.1: Compression jig for testing a single bolt in shear.

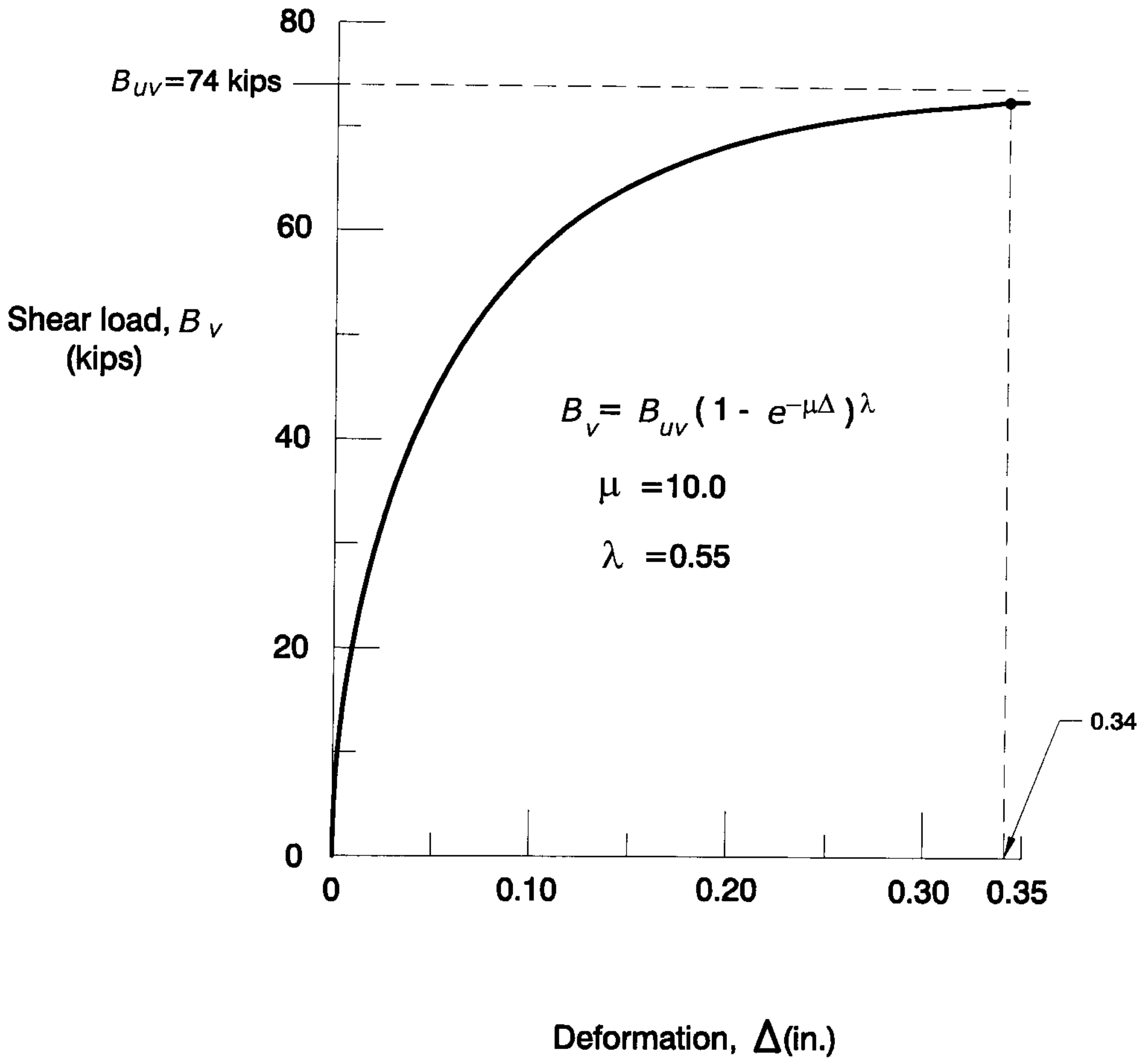


Figure W6.3.2: Load-deformation response of bolts in shear.

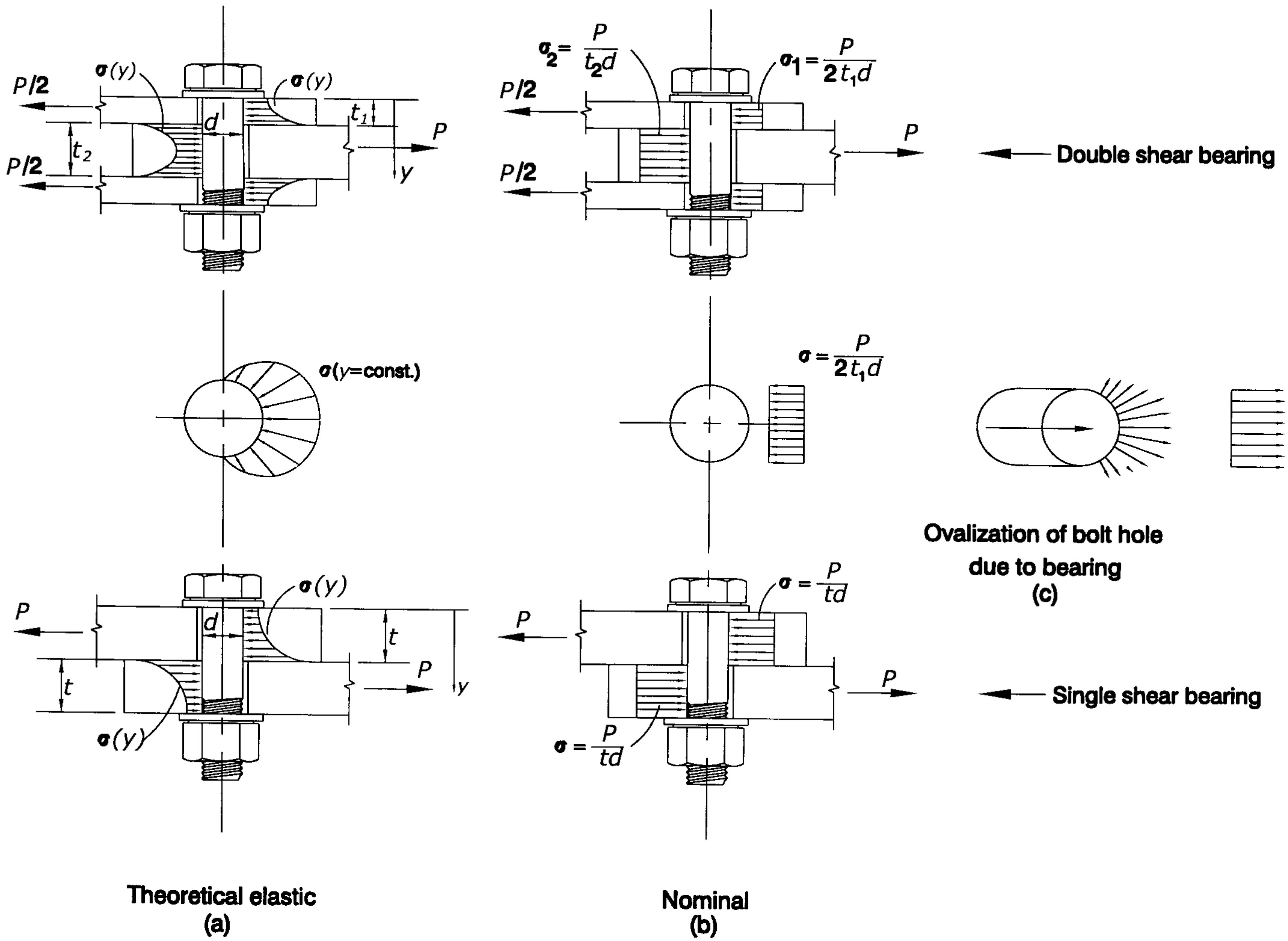


Figure W6.3.3: Bearing stress distrubution in bolted connections.

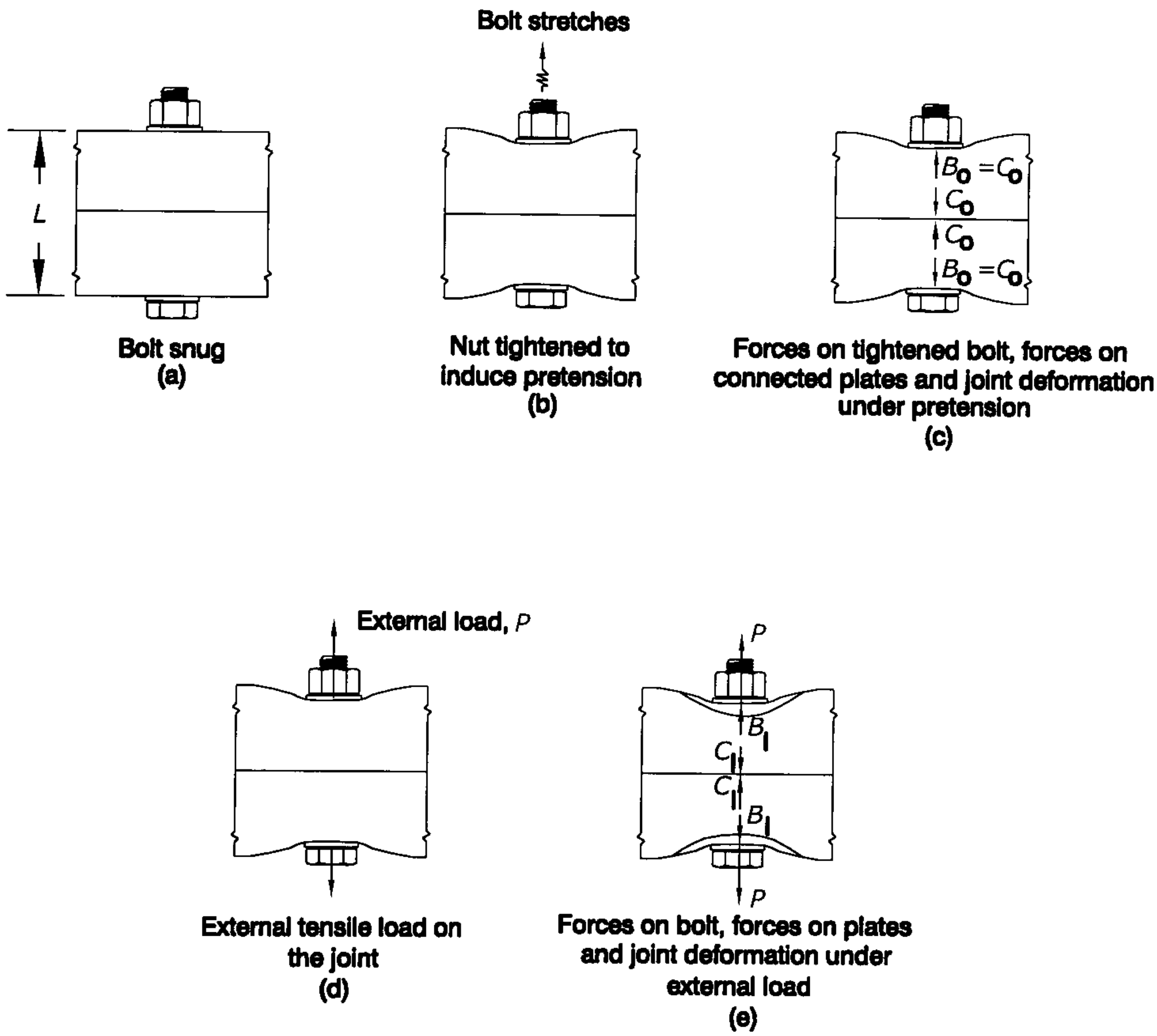
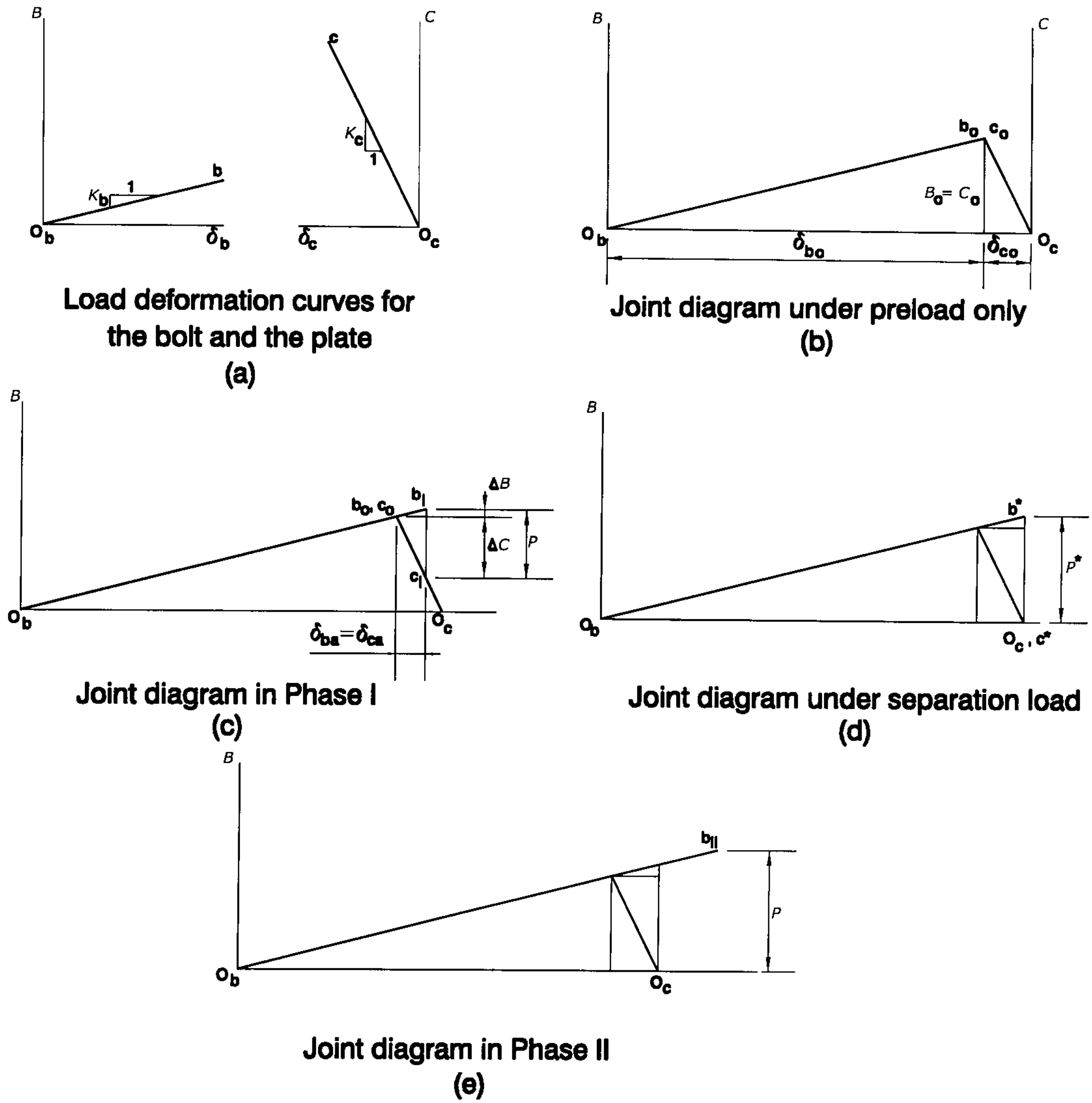


Figure W6.6.1: Single bolt, precompressed joint under tension load.



Load deformation curves for the bolt and the plate
(a)

Joint diagram under preload only
(b)

Joint diagram in Phase I
(c)

Joint diagram under separation load
(d)

Joint diagram in Phase II
(e)

Figure W6.6.2: Force - elongation relationships of a pretensioned joint in tension.

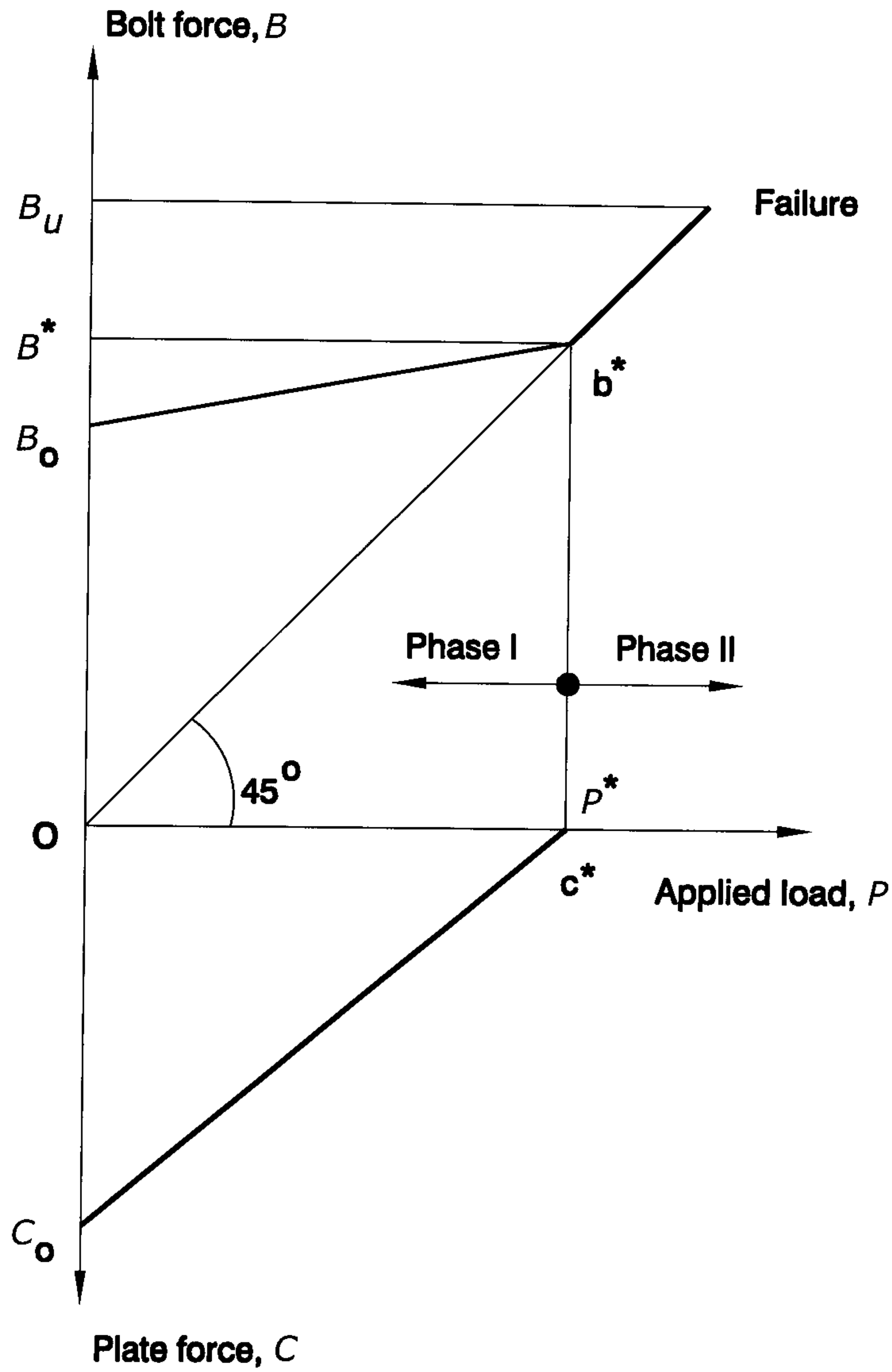


Figure W6.6.3: Bolt force and plate force vs. applied load.

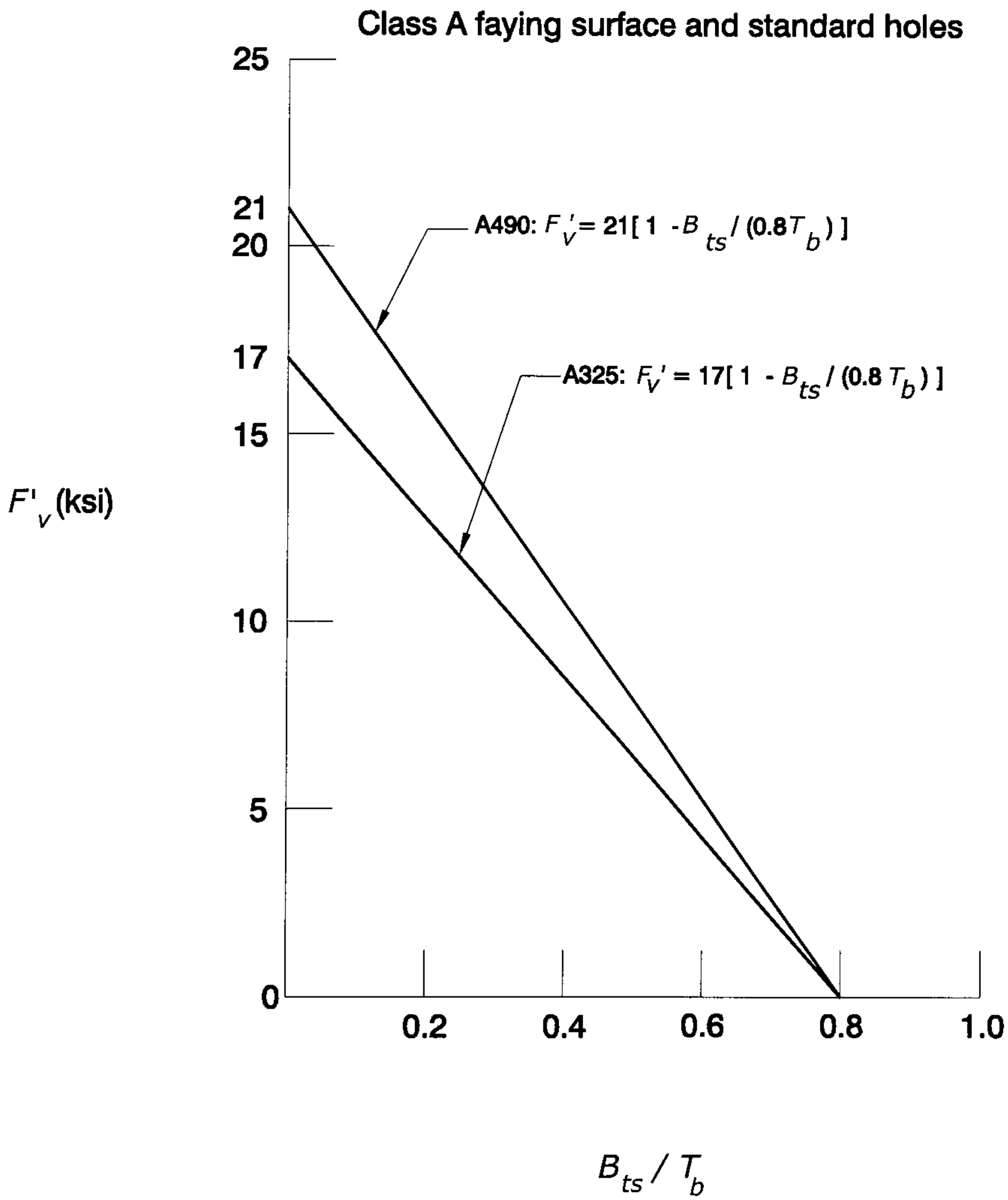


Figure W6.7.1: Combined tension and shear of bolts in slip-critical joints (at the service-load level).

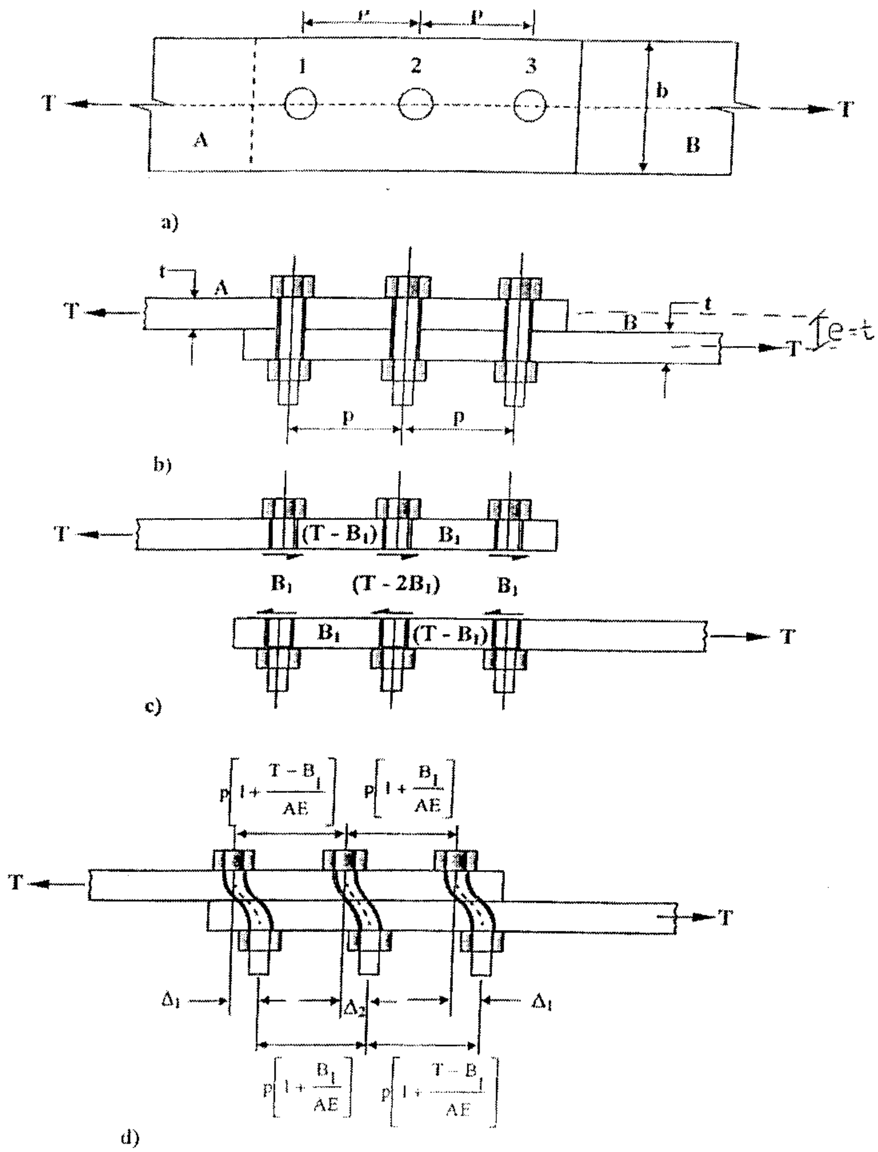
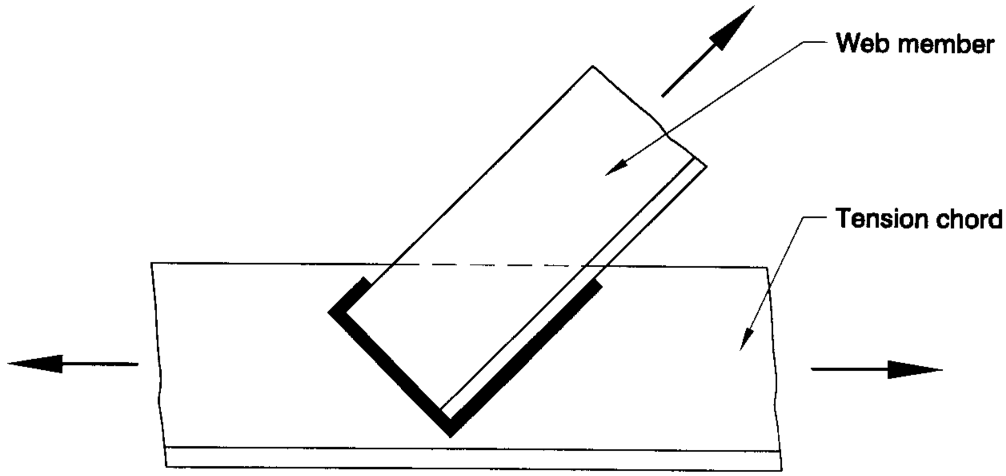
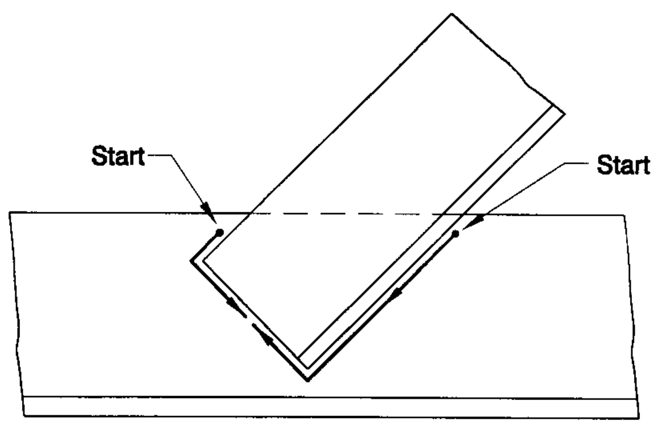


Figure W 6.8.1: Force Distribution in An Axially Loaded Bolted Joint



Fillet welds near tension edges

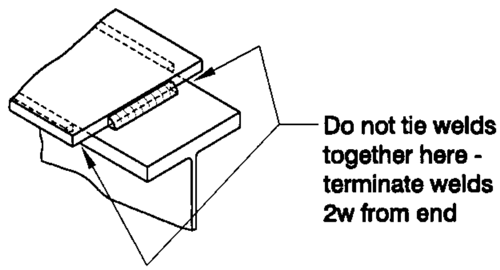
(a)



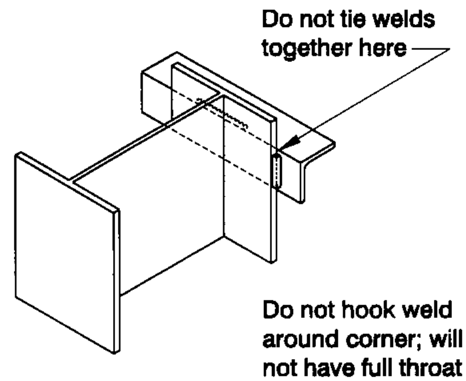
Suggested direction of welding travel to avoid notches

(b)

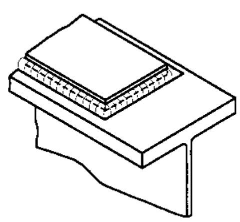
Figure W6.9.1: Fillet weld terminations.



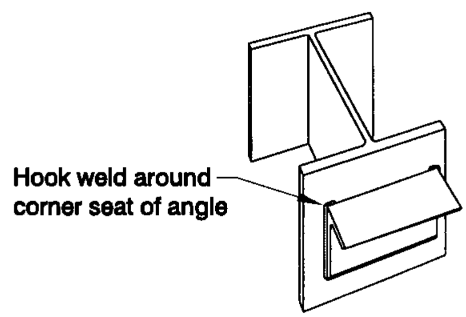
Cover plate width greater
than flange width
(a)



Seat angle width greater
than column flange
(b)



Cover plate width less
than flange width
(c)



Seat angle width less
than column flange
(d)

Figure W6.9.2: Situations where end returns are prohibited.

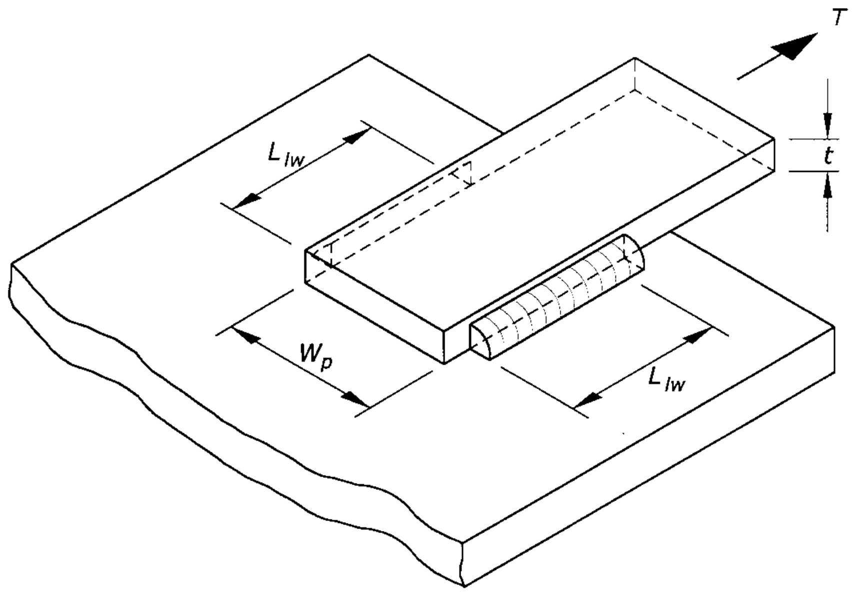


Figure W6.9.3: Flat bar tension member with longitudinal fillet welds only.

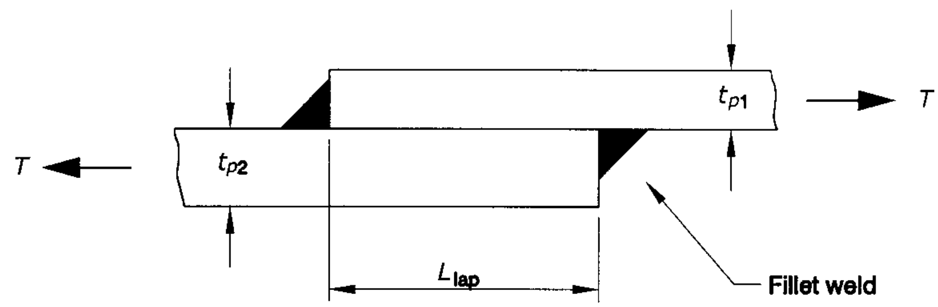


Figure W6.9.4: Overlap of lap joints.

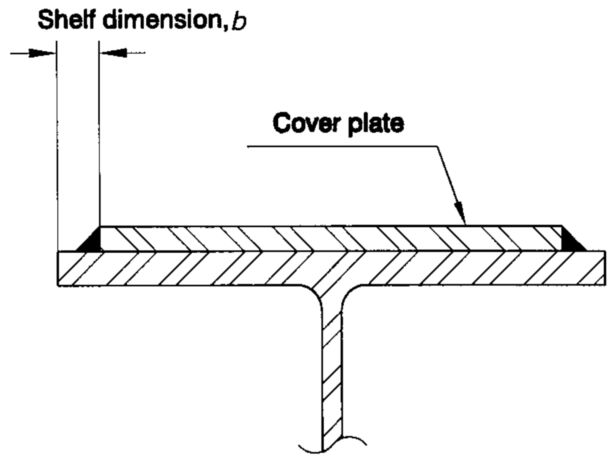
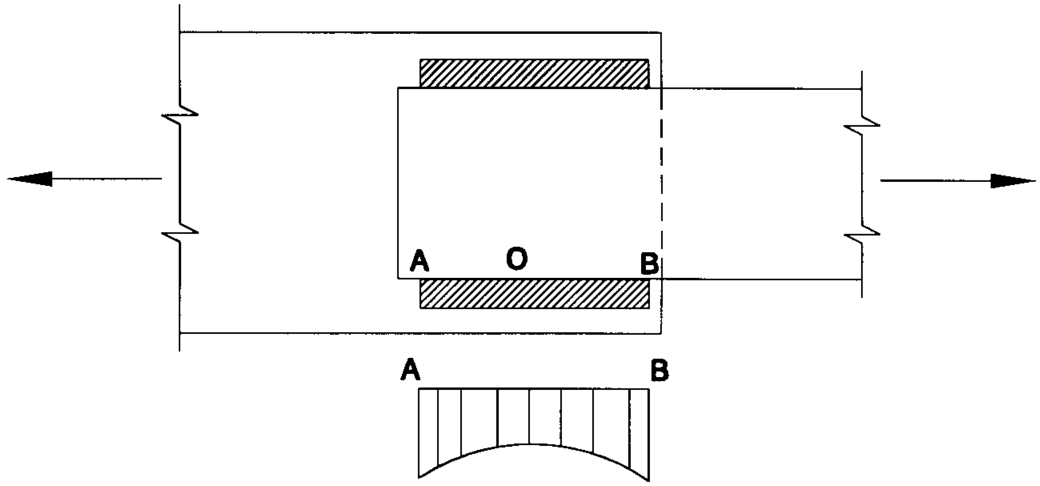
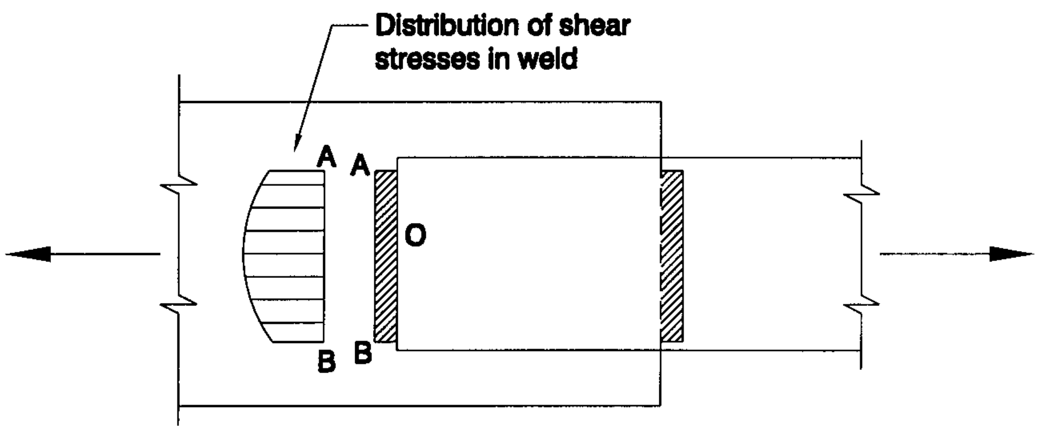


Figure W6.9.5: Shelf dimensions for fillet welds.



Lap joint with longitudinal fillet welds

(a)



Lap joint with transverse fillet welds

(b)

Figure W6.10.1: Distribution of stresses along fillet welds.

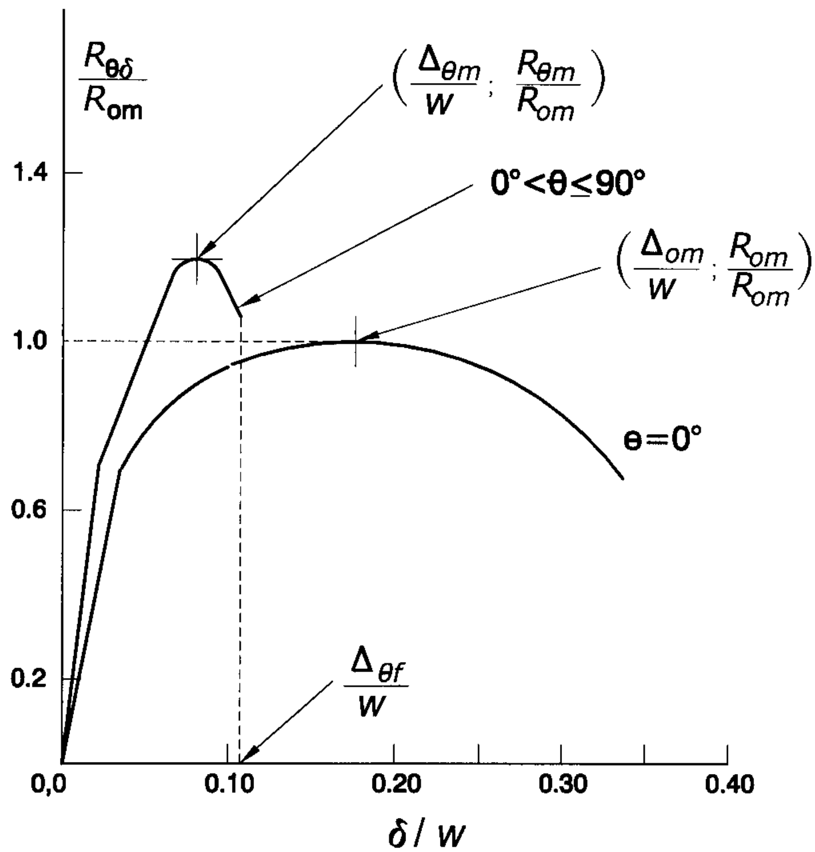


Figure W6.10.2: Load-deformation response of unit length fillet welds.

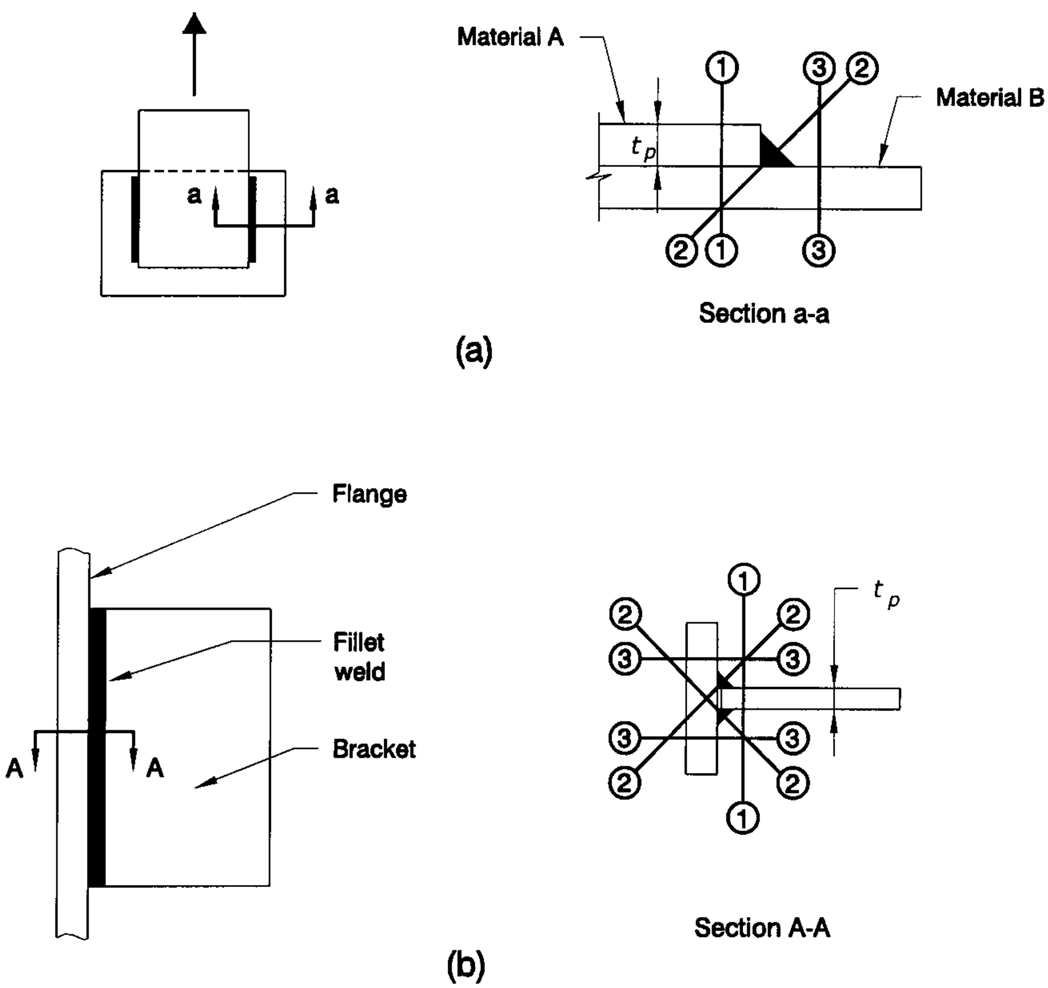


Figure W6.11.1: Critical shear planes for fillet welds loaded in longitudinal shear.

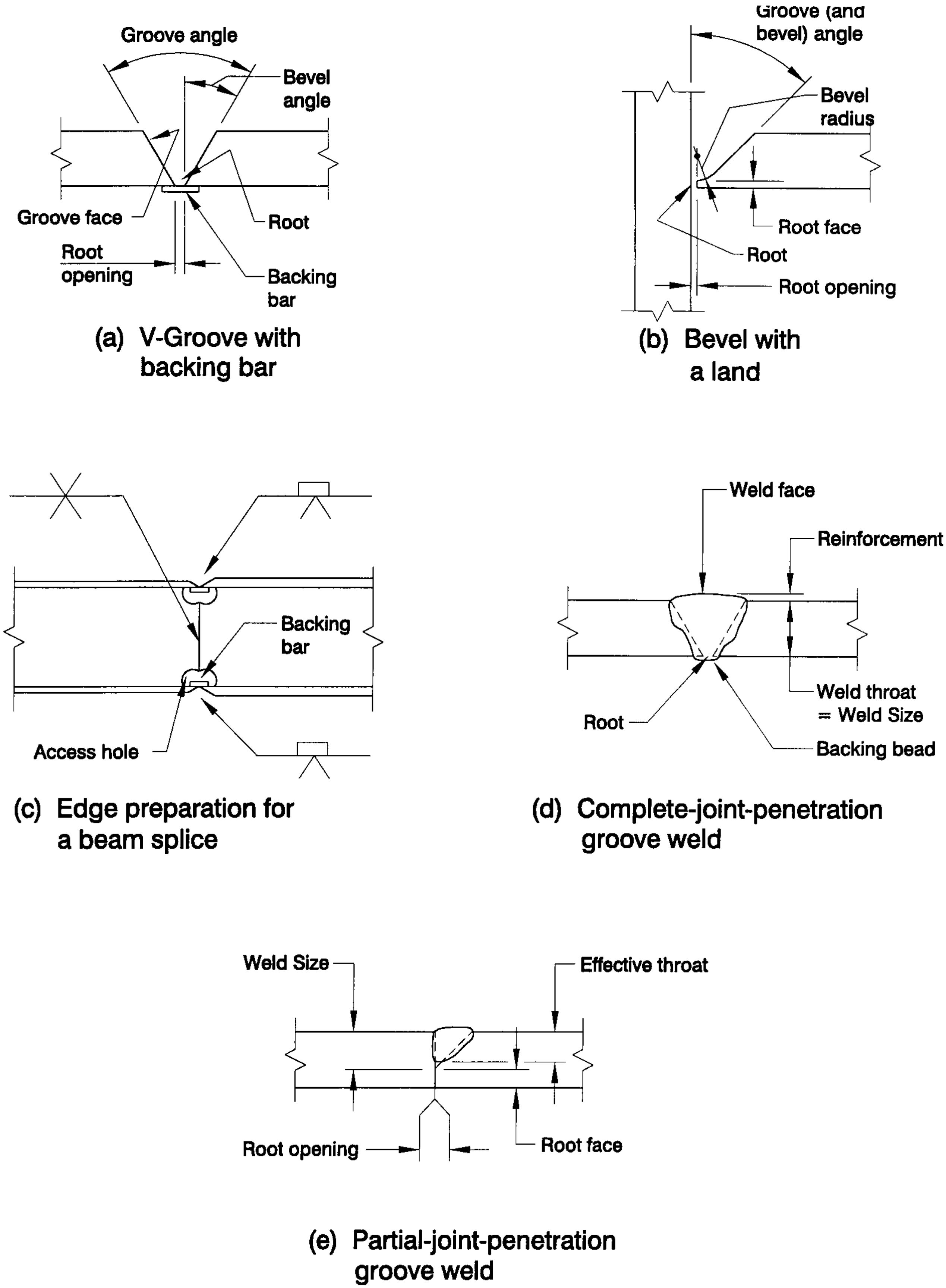


Figure W6.12.1: Nomenclature of groove welds.