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Structural Steel Design[1]

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3.1 Materials

3.1.1 Stress-Strain Behavior of Structural Steel

Structural steel is an important construction material. It possesses attributes such as *strength, stiffness, toughness,* and *ductility* that are very desirable in modern constructions. Strength is the ability of a material to resist stresses. It is measured in terms of the material's yield strength, F_y, and ultimate or tensile strength, F_u. For steel, the ranges of F_y and F_u ordinarily used in constructions are 36 to 50 ksi (248 to 345 MPa) and 58 to 70 ksi (400 to 483 MPa), respectively, although higher strength steels are becoming more common. Stiffness is the ability of a material to resist deformation. It is measured as the slope of the material's stress-strain curve. With reference to Figure 3.1 in which uniaxial engineering stress-strain curves obtained from coupon tests for various grades of steels are shown, it is seen that the modulus of elasticity, E, does not vary appreciably for the different steel grades. Therefore, a value of 29,000 ksi (200 GPa) is often used for design. Toughness is the ability of
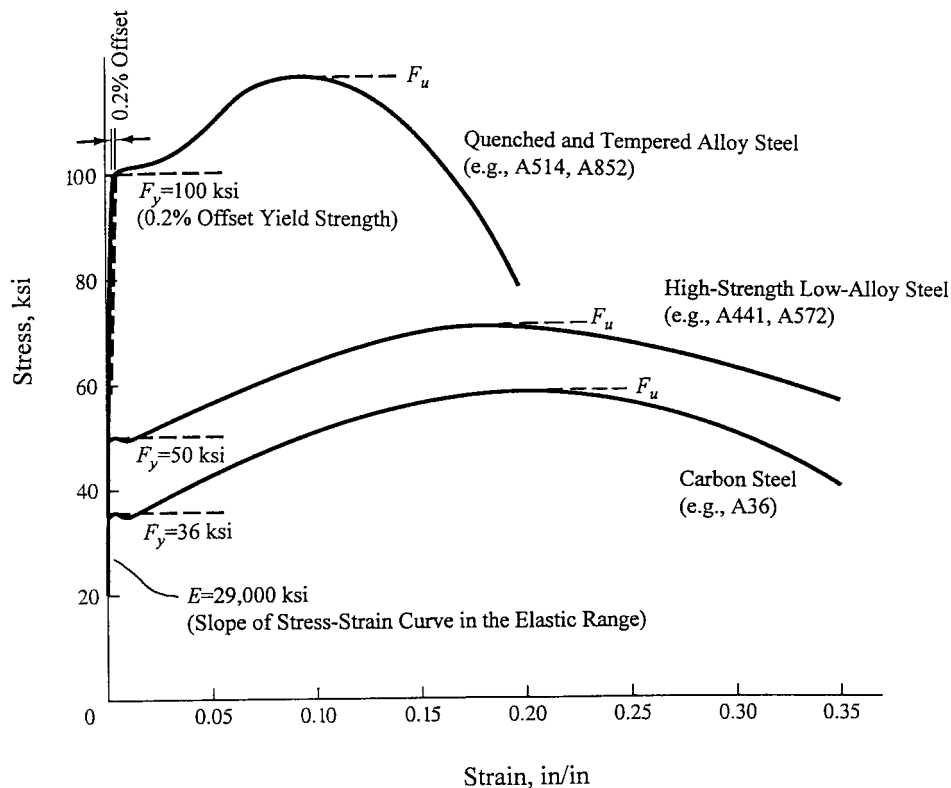
FIGURE 3.1: Uniaxial stress-strain behavior of steel.

a material to absorb energy before failure. It is measured as the area under the material's stress-strain curve. As shown in Figure 3.1, most (especially the lower grade) steels possess high toughness which is suitable for both static and seismic applications. Ductility is the ability of a material to undergo large inelastic, or plastic, deformation before failure. It is measured in terms of percent elongation or percent reduction in area of the specimen tested in uniaxial tension. For steel, percent elongation

ranges from around 10 to 40 for a 2-in. (5-cm) gage length specimen. Ductility generally decreases with increasing steel strength. Ductility is a very important attribute of steel. The ability of structural steel to deform considerably before failure by fracture allows an indeterminate structure to undergo stress redistribution. Ductility also enhances the energy absorption characteristic of the structure, which is extremely important in seismic design.

3.1.2 Types of Steel

Structural steels used for construction purpose are generally grouped into several major American Society of Testing and Materials (ASTM) classifications:

Carbon Steels (ASTM A36, ASTM A529, ASTM 709)

In addition to iron, the main ingredients of this category of steels are carbon (maximum content = 1.7%) and manganese (maximum content = 1.65%), with a small amount (< 0.6%) of silicon and copper. Depending on the amount of carbon content, different types of carbon steels can be identified:

Low carbon steel–carbon content < 0.15%

Mild carbon steel–carbon content varies from 0.15 to 0.29%

Medium carbon steel–carbon content 0.30 to 0.59%

High carbon steel–carbon content 0.60 to 1.70%

The most commonly used structural carbon steel has a mild carbon content. It is extremely ductile and is suitable for both bolting and welding. ASTM A36 is used mainly for buildings. ASTM A529 is occasionally used for bolted and welded building frames and trusses. ASTM 709 is used primarily for bridges.

High Strength Low Alloy Steels (ASTM A441, ASTM A572)

These steels possess enhanced strength as a result of the presence of one or more alloying agents such as chromium, copper, nickel, silicon, vanadium, and others in addition to the basic elements of iron, carbon, and manganese. Normally, the total quantity of all the alloying elements is below 5% of the total composition. These steels generally have higher corrosion-resistant capability than carbon steels. A441 steel was discontinued in 1989; it is superseded by A572 steel.

Corrosion-Resistant High Strength Low Alloy Steels (ASTM A242, ASTM A588)

These steels have enhanced corrosion-resistant capability because of the addition of copper as an alloying element. Corrosion is severely retarded when a layer of patina (an oxidized metallic film) is formed on the steel surfaces. The process of oxidation normally takes place within 1 to 3 years and is signified by a distinct appearance of a deep reddish-brown to black coloration of the steel. For the process to take place, the steel must be subjected to a series of wetting-drying cycles. These steels, especially ASTM 588, are used primarily for bridges and transmission towers (in lieu of galvanized steel) where members are difficult to access for periodic painting.

Quenched and Tempered Alloy Steels (ASTM A852, ASTM A514, ASTM A709, ASTM A852)

The quantities of alloying elements used in these steels are in excess of those used in carbon and low alloy steels. In addition, they are heat treated by quenching and tempering to enhance their strengths. These steels do not exhibit well-defined yield points. Their yield stresses are determined by the 0.2% offset strain method. These steels, despite their enhanced strength, have reduced ductility

(Figure 3.1) and care must be exercised in their usage as the design limit state for the structure or structural elements may be governed by serviceability considerations (e.g., deflection, vibration) and/or local buckling (under compression).
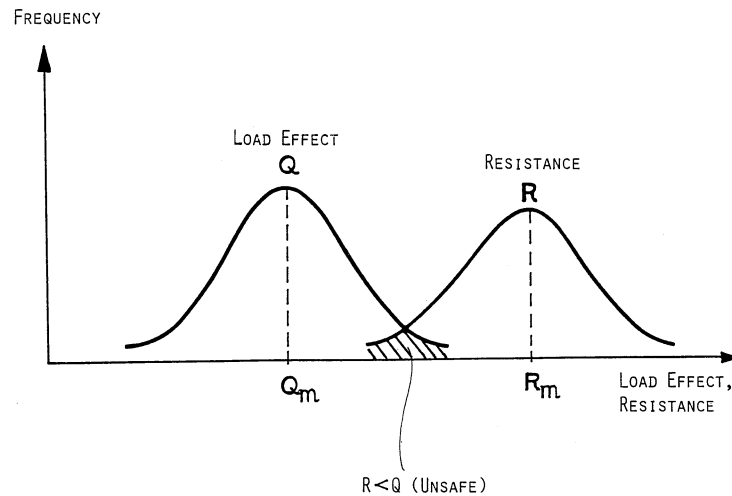

FIGURE 3.2: Frequency distribution of load effect and resistance.

In recent years, a new high strength steel produced using the *thermal-mechanical control process* (TMCP) has been developed. Compared with other high strength steels, TMCP steel has been shown to possess higher strength (for a given carbon equivalent value), enhanced toughness, improved weldability, and lower yield-to-tensile strength ratio, F_y/F_u. A low F_y/F_u value is desirable because there is an inverse relationship between F_y/F_u of the material and rotational capacity of the member. Research on TMCP steel is continuing and, as of this writing, TMCP steel has not been given an ASTM designation.

A summary of the specified minimum yield stresses, F_y, the specified minimum tensile strengths, F_u, and general usages for these various categories of steels are given in Table 3.1.

3.1.3 Fireproofing of Steel

Although steel is an incombustible material, its strength (F_y, F_u) and stiffness (E) reduce quite noticeably at temperatures normally reached in fires when other materials in a building burn. Exposed steel members that will be subjected to high temperature when a fire occurs should be fireproofed to conform to the fire ratings set forth in city codes. Fire ratings are expressed in units of time (usually hours) beyond which the structural members under a standard ASTM Specification (E119) fire test will fail under a specific set of criteria. Various approaches are available for fireproofing steel members. Steel members can be fireproofed by encasement in concrete if a minimum cover of 2 in. (51 mm) of concrete is provided. If the use of concrete is undesirable (because it adds weight to the structure), a lath and plaster (gypsum) ceiling placed underneath the structural members supporting the floor deck of an upper story can be used. In lieu of such a ceiling, spray-on materials such as mineral fibers, perlite, vermiculite, gypsum, etc. can also be used for fireproofing. Other means of fireproofing include placing steel members away from the source of heat, circulating liquid coolant inside box or tubular members and the use of insulative paints. These special paints foam

TABLE 3.1 Types of Steels

ASTM designation	F_y (ksi)[a]	F_u (ksi)[a]	Plate thickness (in.)[b]	General usages
A36	36	58-80	To 8	Riveted, bolted, and welded buildings and bridges.
A529	42	60-85	To 0.5	Similar to A36. The higher yield
	50	70-100	To 1.5	stress for A529 steel allows for savings in weight. A529 supersedes A441.
A572 Grade 42	42	60	To 6	Similar to A441. Grades 60 and 65
Grade 50	50	65	To 4	not suitable for welded bridges.
Grade 60	60	75	To 1.25	
Grade 65	65	80	To 1.25	
A242	42	63	1.5 to 5	Riveted, bolted, and
	46	67	0.75 to 1.5	welded buildings and bridges.
	50	70	0.5 to 0.75	Used when weight savings and enhanced atmospheric corrosion resistance are desired. Specific instructions must be provided for welding.
A588	42	63	5 to 8	Similar to A242. Atmospheric
	46	67	4 to 5	corrosion resistance is about
	50	70	To 4	four times that of A36 steel.
A709 Grade 36	36	58-80	To 4	Primarily for use in bridges.
Grade 50	50	65	To 4	
Grade 50W	50	70	To 4	
Grade 70W	70	90-110	To 4	
Grade 100 & 100W	90	100-130	2.5 to 4	
Grade 100 & 100W	100	110-130	To 2.5	
A852	70	90-110	To 4	Plates for welded and bolted construction where atmospheric corrosion resistance is desired.
A514	90-100	100-130	2.5 to 6	Primarily for welded bridges. Avoid
		110-130		usage if ductility is important.

[a] 1 ksi = 6.895 MPa
[b] 1 in. = 25.4 mm

and expand when heated, thus forming a shield for the members [26]. For a more detailed discussion of structural steel design for fire protection, refer to the latest edition of AISI publication No. FS3, *Fire-Safe Structural Steel-A Design Guide.* Additional information on fire-resistant standards and fire protection can be found in the AISI booklets on *Fire Resistant Steel Frame Construction, Designing Fire Protection for Steel Columns,* and *Designing Fire Protection for Steel Trusses* as well as in the *Uniform Building Code.*

3.1.4 Corrosion Protection of Steel

Atmospheric corrosion occurs when steel is exposed to a continuous supply of water and oxygen. The rate of corrosion can be reduced if a barrier is used to keep water and oxygen from contact with the surface of bare steel. Painting is a practical and cost effective way to protect steel from corrosion. The Steel Structures Painting Council issues specifications for the surface preparation and the painting of steel structures for corrosion protection of steel. In lieu of painting, the use of other coating materials such as epoxies or other mineral and polymeric compounds can be considered. The use of corrosion resistance steel such as ASTM A242 and A588 steel or galvanized steel is another alternative.

3.1.5 Structural Steel Shapes

Steel sections used for construction are available in a variety of shapes and sizes. In general, there are three procedures by which steel shapes can be formed: hot-rolled, cold-formed, and welded. All steel shapes must be manufactured to meet ASTM standards. Commonly used steel shapes include the wide flange (W) sections, the American Standard beam (S) sections, bearing pile (HP) sections, American Standard channel (C) sections, angle (L) sections, and tee (WT) sections as well as bars,

plates, pipes, and tubular sections. H sections which, by dimensions, cannot be classified as W or S shapes are designated as miscellaneous (M) sections, and C sections which, by dimensions, cannot be classified as American Standard channels are designated as miscellaneous channel (MC) sections.

Hot-rolled shapes are classified in accordance with their tensile property into five size groups by the American Society of Steel Construction (AISC). The groupings are given in the AISC Manuals [21, 22] Groups 4 and 5 shapes and group 3 shapes with flange thickness exceeding 1-1/2 in. are generally used for application as compression members. When weldings are used, care must be exercised to minimize the possibility of cracking in regions at the vicinity of the welds by carefully reviewing the material specification and fabrication procedures of the pieces to be joined.

3.1.6 Structural Fasteners

Steel sections can be fastened together by rivets, bolts, and welds. While rivets were used quite extensively in the past, their use in modern steel construction has become almost obsolete. Bolts have essentially replaced rivets as the primary means to connect nonwelded structural components.

Bolts

Four basic types of bolts are commonly in use. They are designated by ASTM as A307, A325, A490, and A449. A307 bolts are called unfinished or ordinary bolts. They are made from low carbon steel. Two grades (A and B) are available. They are available in diameters from 1/4 in. to 4 in. in 1/8 in. increments. They are used primarily for low-stress connections and for secondary members. A325 and A490 bolts are called high-strength bolts. A325 bolts are made from a heat-treated medium carbon steel. They are available in three types: Type 1—bolts made of medium carbon steel; Type 2—bolts made of low carbon martensite steel; and Type 3—bolts having atmospheric-corrosion resistance and weathering characteristics comparable to A242 and A588 steel. A490 bolts are made from quenched and tempered alloy steel and thus have a higher strength than A325 bolts. Like A325 bolts, three types (Types 1 to 3) are available. Both A325 and A490 bolts are available in diameters from 1/2 in. to 1-1/2 in. in 1/8 in. increments. They are used for general construction purposes. A449 bolts are made from quenched and tempered steel. They are available in diameters from 1/4 in. to 3 in. A449 bolts are used when diameters over 1-1/2 in. are needed. They are also used for anchor bolts and threaded rod.

High-strength bolts can be tightened to two conditions of tightness: snug-tight and fully tight. Snug-tight conditions can be attained by a few impacts of an impact wrench, or the full effort of a worker using an ordinary spud wrench. Snug-tight conditions must be clearly identified on the design drawing and are permitted only if the bolts are not subjected to tension loads, and loosening or fatigue due to vibration or load fluctuations are not design considerations. Bolts used in slip-critical conditions (i.e., conditions for which the integrity of the connected parts is dependent on the frictional force developed between the interfaces of the joint) and in conditions where the bolts are subjected to direct tension are required to be fully tightened to develop a pretension force equal to about 70% of the minimum tensile stress F_u of the material from which the bolts are made. This can be accomplished by using the turn-of-the-nut method, the calibrated wrench method, or by the use of alternate design fasteners or direct tension indicator [28].

Welds

Welding is a very effective means to connect two or more pieces of material together. The four most commonly used welding processes are *Shielded Metal Arc Welding* (SMAW), *Submerged Arc Welding* (SAW), *Gas Metal Arc Welding* (GMAW), and *Flux Core Arc Welding* (FCAW) [7]. Welding can be done with or without filler materials although most weldings used for construction utilized filler materials. The filler materials used in modern day welding processes are electrodes. Table 3.2

summarizes the electrode designations used for the aforementioned four most commonly used welding processes.

TABLE 3.2 Electrode Designations

Welding processes	Electrode designations	Remarks
Shielded metal arc welding (SMAW)	E60XX E70XX E80XX E100XX E110XX	The 'E' denotes electrode. The first two digits indicate tensile strength in ksi.[a] The two 'X's represent numbers indicating the usage of the electrode.
Submerged arc welding (SAW)	F6X-EXXX F7X-EXXX F8X-EXXX	The 'F' designates a granular flux material. The digit(s) following the 'F' indicate the tensile strength in ksi (6 means 60 ksi, 10 means 100 ksi, etc.).
	F10X-EXXX F11X-EXXX	The digit before the hyphen gives the Charpy V-notched impact strength. The 'E' and the 'X's that follow represent numbers relating to the use of the electrode.
Gas metal arc welding (GMAW)	ER70S-X ER80S ER100S ER110S	The digits following the letters 'ER' represent the tensile strength of the electrode in ksi.
Flux cored arc welding (FCAW)	E6XT-X E7XT-X E8XT E10XT E11XT	The digit(s) following the letter 'E' represent the tensile strength of the electrode in ksi (6 means 60 ksi, 10 means 100 ksi, etc.).

[a] 1 ksi = 6.895 MPa

Finished welds should be inspected to ensure their quality. Inspection should be performed by qualified welding inspectors. A number of inspection methods are available for weld inspections. They include visual, the use of liquid penetrants, magnetic particles, ultrasonic equipment, and radiographic methods. Discussion of these and other welding inspection techniques can be found in the *Welding Handbook* [6].

3.1.7 Weldability of Steel

Most ASTM specification construction steels are weldable. In general, the strength of the electrode used should equal or exceed the strength of the steel being welded [7]. The table below gives ranges of chemical elements in steel within which good weldability is assured [8].

Element	Range for good weldability	Percent requiring special care
Carbon	0.06-0.25	0.35
Manganese	0.35-0.80	1.40
Silicon	0.10 max.	0.30
Sulfur	0.035 max.	0.050
Phosphorus	0.030 max.	0.040

Weldability of steel is closely related to the amount of carbon in steel. Weldability is also affected by the presence of other elements. A quantity known as *carbon equivalent value,* giving the amount of carbon and other elements in percent composition, is often used to define the chemical requirements in steel. One definition of the carbon equivalent value C_{eq} is

$$C_{eq} = \text{Carbon} + \frac{(\text{Manganese} + \text{Silicon})}{6} + \frac{(\text{Copper} + \text{Nickel})}{15}$$
$$+ \frac{(\text{Chromium} + \text{Molybdenum} + \text{Vanadium} + \text{Columbium})}{5} \tag{3.1}$$

A steel is considered weldable if $C_{eq} \leq 0.50\%$ for steel in which the carbon content does not exceed 0.12%, and if $C_{eq} \leq 0.45\%$ for steel in which the carbon content exceeds 0.12%.

3.2 Design Philosophy and Design Formats

3.2.1 Design Philosophy

Structural design should be performed to satisfy three criteria: (1) strength, (2) serviceability, and (3) economy. *Strength* pertains to the general integrity and safety of the structure under extreme load conditions. The structure is expected to withstand occasional overloads without severe distress and damage during its lifetime. *Serviceability* refers to the proper functioning of the structure as related to its appearance, maintainability, and durability under normal, or service load, conditions. Deflection, vibration, permanent deformation, cracking, and corrosion are some design considerations associated with serviceability. *Economy* concerns the overall material and labor costs required for the design, fabrication, erection, and maintenance processes of the structure.

3.2.2 Design Formats

At present, steel design can be performed in accordance with one of the following three formats:

1. *Allowable Stress Design (ASD)*— ASD has been in use for decades for steel design of buildings and bridges. It continues to enjoy popularity among structural engineers engaged in steel building design. In allowable stress (or working stress) design, member stresses computed under the action of service (or working) loads are compared to some predesignated stresses called allowable stresses. The allowable stresses are usually expressed as a function of the yield stress (F_y) or tensile stress (F_u) of the material. To account for overload, understrength, and approximations used in structural analysis, a factor of safety is applied to reduce the nominal resistance of the structural member to a fraction of its tangible capacity. The general format for an allowable stress design has the form

$$\frac{R_n}{F.S.} \geq \sum_{i=1}^{m} Q_{ni} \qquad (3.2)$$

   where R_n is the nominal resistance of the structural component expressed in a unit of stress; Q_{ni} is the service, or working stresses computed from the applied working load of type i; $F.S.$ is the factor of safety; i is the load type (dead, live, wind, etc.), and m is the number of load type considered in the design. The left-hand side of the equation, $R_n/F.S.$, represents the allowable stress of the structural component.

2. *Plastic Design (PD)*— PD makes use of the fact that steel sections have reserved strength beyond the first yield condition. When a section is under flexure, yielding of the cross-section occurs in a progressive manner, commencing with the fibers farthest away from the neutral axis and ending with the fibers nearest the neutral axis. This phenomenon of progressive yielding, referred to as *plastification,* means that the cross-section does not fail at first yield. The additional moment that a cross-section can carry in excess of the moment that corresponds to first yield varies depending on the shape of the cross-section. To quantify such reserved capacity, a quantity called *shape factor,* defined as the ratio of the *plastic moment* (moment that causes the entire cross-section to yield, resulting in the formation of a plastic hinge) to the *yield moment* (moment that causes yielding of the extreme fibers only) is used. The shape factor for hot-rolled I-shaped sections bent about

the strong axes has a value of about 1.15. The value is about 1.50 when these sections are bent about their weak axes.

For an indeterminate structure, failure of the structure will not occur after the formation of a plastic hinge. After complete yielding of a cross-section, force (or, more precisely, moment) redistribution will occur in which the unfailed portion of the structure continues to carry any additional loadings. Failure will occur only when enough cross-sections have yielded rendering the structure unstable, resulting in the formation of a *plastic collapse mechanism.*

In plastic design, the factor of safety is applied to the applied loads to obtain factored loads. A design is said to have satisfied the strength criterion if the load effects (i.e., forces, shears, and moments) computed using these factored loads do not exceed the nominal plastic strength of the structural component. Plastic design has the form

$$R_n \geq \gamma \sum_{i=1}^{m} Q_{ni} \tag{3.3}$$

where R_n is the nominal plastic strength of the member; Q_{ni} is the nominal load effect from loads of type i; γ is the load factor; i is the load type; and m is the number of load types.

In steel building design, the load factor is given by the AISC Specification as 1.7 if Q_n consists of dead and live gravity loads only, and as 1.3 if Q_n consists of dead and live gravity loads acting in conjunction with wind or earthquake loads.

3. *Load and Resistance Factor Design (LRFD)*— LRFD is a probability-based limit state design procedure. In its development, both load effects and resistance were treated as random variables. Their variabilities and uncertainties were represented by frequency distribution curves. A design is considered satisfactory according to the strength criterion if the resistance exceeds the load effects by a comfortable margin. The concept of safety is represented schematically in Figure 3.2. Theoretically, the structure will not fail unless R is less than Q as shown by the shaded portion in the figure where the R and Q curves overlap. The smaller this shaded area, the less likely that the structure will fail. In actual design, a resistance factor ϕ is applied to the nominal resistance of the structural component to account for any uncertainties associated with the determination of its strength and a load factor γ is applied to each load type to account for the uncertainties and difficulties associated with determining its actual load magnitude. Different load factors are used for different load types to reflect the varying degree of uncertainty associated with the determination of load magnitudes. In general, a lower load factor is used for a load that is more predicable and a higher load factor is used for a load that is less predicable. Mathematically, the LRFD format takes the form

$$\phi R_n \geq \sum_{i=1}^{m} \gamma_i Q_{ni} \tag{3.4}$$

where ϕR_n represents the design (or usable) strength, and $\Sigma \gamma Q_{ni}$ represents the required strength or load effect for a given load combination. Table 3.3 shows the load combinations to be used on the right hand side of Equation 3.4. For a safe design, all load combinations should be investigated and the design is based on the worst case scenario.

LRFD is based on the limit state design concept. A limit state is defined as a condition in which a structure or structural component becomes unsafe (that is, a violation of the

strength limit state) or unsuitable for its intended function (that is, a violation of the serviceability limit state). In a limit state design, the structure or structural component is designed in accordance to its limits of usefulness, which may be strength related or serviceability related.

TABLE 3.3 Load Factors and Load Combinations

$1.4D$
$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W)$
$1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2D \pm 1.0E + 0.5L + 0.2S$
$0.9D \pm (1.3W \text{ or } 1.0E)$

where

D	$=$	dead load
L	$=$	live load
L_r	$=$	roof live load
W	$=$	wind load
S	$=$	snow load
E	$=$	earthquake load
R	$=$	nominal load due to initial rainwater or ice exclusive of the ponding contribution

The load factor on L in the third, fourth, and fifth load combinations shown above shall equal 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf (47.9 N/m^2).

3.3 Tension Members

Tension members are to be designed to preclude the following possible modes of failures under normal load conditions: Yielding in gross section, fracture in effective net section, block shear, shear rupture along plane through the fasteners, bearing on fastener holes, prying (for lap or hanger-type joints). In addition, the fasteners' strength must be adequate to prevent failure in the fasteners. Also, except for rods in tension, the slenderness of the tension member obtained by dividing the length of the member by its least radius of gyration should preferably not exceed 300.

3.3.1 Allowable Stress Design

The computed tensile stress, f_t, in a tension member shall not exceed the allowable stress for tension, F_t, given by $0.60F_y$ for yielding on the gross area, and by $0.50F_u$ for fracture on the effective net area. While the gross area is just the nominal cross-sectional area of the member, the *effective net area* is the smallest cross-sectional area accounting for the presence of fastener holes and the effect of shear lag. It is calculated using the equation

$$
\begin{aligned}
A_e &= U A_n \\
&= U \left[A_g - \sum_{i=1}^{m} d_{ni} t_i + \sum_{j=1}^{k} \left(\frac{s^2}{4g} \right)_j t_j \right]
\end{aligned}
\tag{3.5}
$$

where

U is a reduction coefficient given by [25]

$$U = 1 - \frac{\bar{x}}{l} \le 0.90 \tag{3.6}$$

in which l is the length of the connection and $\bar{x}$ is the distance measured as shown in Figure 3.3. For a given cross-section the largest $\bar{x}$ is used in Equation 3.6 to calculate U. This reduction coefficient is introduced to account for the shear lag effect that arises when some component elements of the cross-section in a joint are not connected, rendering the connection less effective in transmitting the applied load. The terms in brackets in Equation 3.5 constitute the so-called net section A_n. The

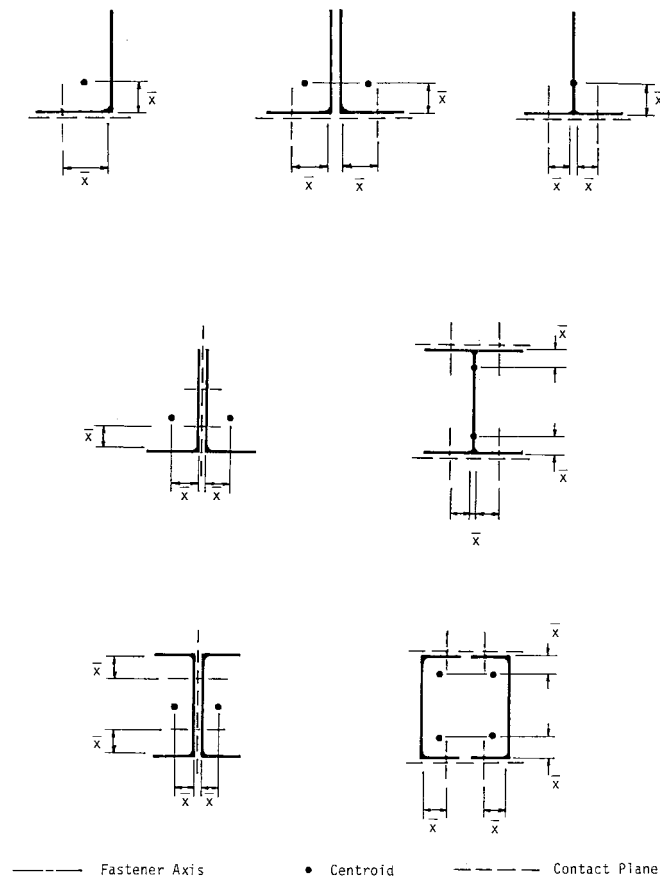FIGURE 3.3: Definition of $\bar{x}$ for selected cross-sections.

various terms are defined as follows:

A_g = gross cross-sectional area

d_n = nominal diameter of the hole (bolt cutout), taken as the nominal bolt diameter plus 1/8 of an inch (3.2 mm)

t = thickness of the component element

s = longitudinal center-to-center spacing (pitch) of any two consecutive fasteners in a chain of staggered holes

g = transverse center-to-center spacing (gage) between two adjacent fasteners gage lines in a chain of staggered holes

The second term inside the brackets of Equation 3.5 accounts for loss of material due to bolt cutouts, the summation is carried for all bolt cutouts lying on the failure line. The last term inside the brackets of Equation 3.5 indirectly accounts for the effect of the existence of a combined stress state (tensile and shear) along an inclined failure path associated with staggered holes. The summation is carried for all staggered paths along the failure line. This term vanishes if the holes are not staggered. Normally, it is necessary to investigate different failure paths that may occur in a connection, the critical failure path is the one giving the smallest value for A_e.

To prevent block shear failure and shear rupture, the allowable stresses for block shear and shear rupture are specified as follows.

Block shear:
$$R_{BS} = 0.30A_v F_u + 0.50A_t F_u \tag{3.7}$$

Shear rupture:
$$F_v = 0.30F_u \tag{3.8}$$

where
A_v = net area in shear
A_t = net area in tension
F_u = specified minimum tensile strength

The tension member should also be designed to possess adequate thickness and the fasteners should be placed within a specific range of spacings and edge distances to prevent failure due to bearing and failure by prying action (see section on Connections).

3.3.2   Load and Resistance Factor Design

According to the LRFD Specification [18], tension members designed to resist a factored axial force of P_u calculated using the load combinations shown in Table 3.3 must satisfy the condition of

$$\phi_t P_n \geq P_u \tag{3.9}$$

The design strength $\phi_t P_n$ is evaluated as follows.

Yielding on gross section:
$$\phi_t P_n = 0.90[F_y A_g] \tag{3.10}$$

where
0.90 = the resistance factor for tension
F_y = the specified minimum yield stress of the material
A_g = the gross cross-sectional area of the member

Fracture in effective net section:
$$\phi_t P_n = 0.75[F_u A_e] \tag{3.11}$$

where
0.75 = the resistance factor for fracture in tension
F_u = the specified minimum tensile strength
A_e = the effective net area given in Equation 3.5

Block shear: If $F_u A_{nt} \geq 0.6 F_u A_{nv}$ (i.e., shear yield-tension fracture)

$$\phi_t P_n = 0.75[0.60 F_y A_{gv} + F_u A_{nt}] \tag{3.12a}$$

If $F_u A_{nt} < 0.6 F_u A_{nv}$ (i.e., shear fracture-tension yield)

$$\phi_t P_n = 0.75[0.60 F_u A_{nv} + F_y A_{gt}] \tag{3.12b}$$

where

0.75	=	the resistance factor for block shear
F_y, F_u	=	the specified minimum yield stress and tensile strength, respectively
A_{gv}	=	the gross area of the torn-out segment subject to shear
A_{nt}	=	the net area of the torn-out segment subject to tension
A_{nv}	=	the net area of the torn-out segment subject to shear
A_{gt}	=	the gross area of the torn-out segment subject to tension

EXAMPLE 3.1:

Using LRFD, select a double channel tension member shown in Figure 3.4a to carry a dead load D of 40 kips and a live load L of 100 kips. The member is 15 feet long. Six 1-in. diameter A325 bolts in standard size holes are used to connect the member to a 3/8-in. gusset plate. Use A36 steel (F_y =36 ksi, F_u =58 ksi) for all the connected parts.

Load Combinations:

From Table 3.3, the applicable load combinations are:

$$1.4D = 1.4(40) = 56 \text{ kips}$$
$$1.2D + 1.6L = 1.2(40) + 1.6(100) = 208 \text{ kips}$$

The design of the tension member is to be based on the larger of the two, i.e., 208 kips and so *each* channel is expected to carry 104 kips.

Yielding in gross section:

Using Equations 3.9 and 3.10, the gross area required to prevent cross-section yielding is

$$
\begin{aligned}
0.90[F_y A_g] &\geq P_u \\
0.90[(36)(A_g)] &\geq 104 \\
(A_g)_{req'd} &\geq 3.21 \text{ in}^2
\end{aligned}
$$

From the section properties table contained in the AISC-LRFD Manual, one can select the following trial sections: C8x11.5 (A_g =3.38 in^2), C9x13.4 (A_g =3.94 in^2), C8x13.75 (A_g =4.04 in^2).

Check for the limit state of fracture on effective net section:

The above sections are checked for the limiting state of fracture in the following table.
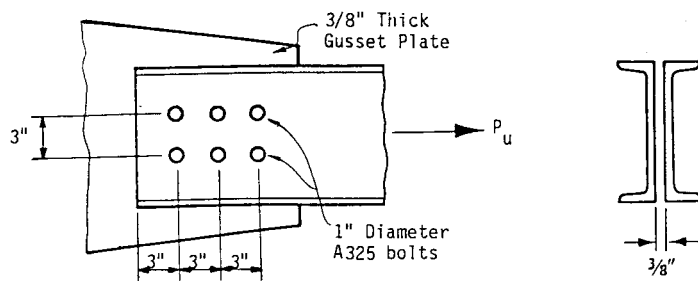
(a) A Double Channel Tension Member
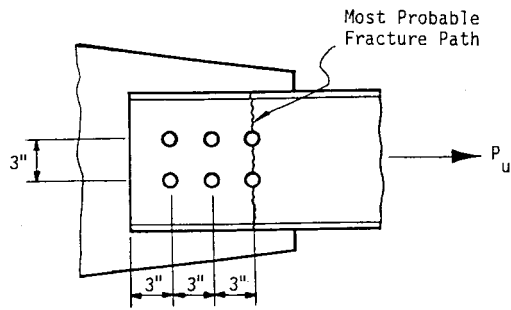
(b) Fracture Failure
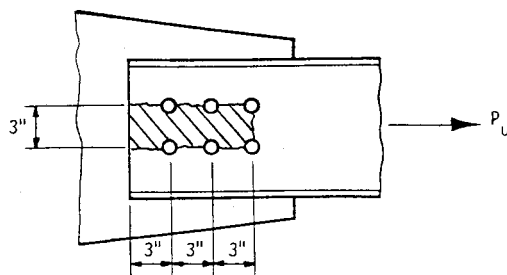
(c) Block Shear Failure

FIGURE 3.4: Design of a double-channel tension member (1 in. = 25.4 mm).

Section	A_g (in.^{2})	t_w (in.)	$\bar{x}$ (in.)	U^a	A_e^b (in.^{2})	$\phi_t P_n$ (kips)
C8x11.5	3.38	0.220	0.571	0.90	2.6	113.1
C9x13.4	3.94	0.233	0.601	0.90	3.07	133.5
C8x13.75	4.04	0.303	0.553	0.90	3.02	131.4

^{a} Equation 3.6

^{b} Equation 3.5, Figure 3.4b

From the last column of the above table, it can be seen that fracture is not a problem for any of the trial section.

Check for the limit state of block shear:

Figure 3.4c shows a possible block shear failure mode. To avoid block shear failure the required strength of $P_u = 104$ kips should not exceed the design strength, $\phi_t P_n$, calculated using Equation 3.12a or Equation 3.12b, whichever is applicable.

For the C8x11.5 section:

$$
\begin{aligned}
A_{gv} &= 2(9)(0.220) = 3.96 \text{ in.}^2 \\
A_{nv} &= A_{gv} - 5(1 + 1/8)(0.220) = 2.72 \text{ in.}^2 \\
A_{gt} &= (3)(0.220) = 0.66 \text{ in.}^2 \\
A_{nt} &= A_{gt} - 1(1 + 1/8)(0.220) = 0.41 \text{ in.}^2
\end{aligned}
$$

Substituting the above into Equations 3.12b since $[0.6 F_u A_{nv} = 94.7 \text{ kips}]$ is larger than $[F_u A_{nt} = 23.8 \text{ kips}]$, we obtain $\phi_t P_n = 88.8$ kips, which is less than $P_u = 104$ kips. The C8x11.5 section is therefore not adequate. Significant increase in block shear strength is not expected from the C9x13.4 section because its web thickness t_w is just slightly over that of the C8x11.5 section. As a result, we shall check the adequacy of the C8x13.75 section instead.

For the C8x13.75 section:

$$
\begin{aligned}
A_{gv} &= 2(9)(0.303) = 5.45 \text{ in.}^2 \\
A_{nv} &= A_{gv} - 5(1 + 1/8)(0.303) = 3.75 \text{ in.}^2 \\
A_{gt} &= (3)(0.303) = 0.91 \text{ in.}^2 \\
A_{nt} &= A_{gt} - 1(1 + 1/8)(0.303) = 0.57 \text{ in.}^2
\end{aligned}
$$

Substituting the above into Equations 3.12b since $[0.6 F_u A_{nv} = 130.5 \text{ kips}]$ is larger than $[F_u A_{nt} = 33.1 \text{ kips}]$ we obtain $\phi_t P_n = 122$ kips, which exceeds the required strength P_u of 104 kips. Therefore, block shear will not be a problem for the C8x13.75 section.

Check for the limiting slenderness ratio:

Using the parallel axis theorem, the least radius of gyration of the double channel cross-section is calculated to be 0.96 in. Therefore, $L/r = (15)(12)/0.96 = 187.5$ which is less than the recommended maximum value of 300.

Check for the adequacy of the connection:

The calculations are shown in an example in the section on Connections.

Longitudinal spacing of connectors:

According to Section J3.5 of the LRFD Specification, the maximum spacing of connectors in built-up tension members shall not exceed:

- 24 times the thickness of the thinner plate or 12 in. for painted members or unpainted members not subject to corrosion.

- 14 times the thickness of the thinner plate or 7 in. for unpainted members of weathering steel subject to atmospheric corrosion.

Assuming the first condition applies, a spacing of 6 in. is to be used.

Use 2C8x13.75 Connected Intermittently at 6-in. Interval

3.3.3 Pin-Connected Members

Pin-connected members shall be designed to preclude the following modes of failure: (1) tension yielding on the gross area; (2) tension fracture on the effective net area; (3) longitudinal shear on the effective area; and (4) bearing on the projected pin area (Figure 3.5).

Allowable Stress Design

The allowable stresses for tension yield, tension fracture, and shear rupture are $0.60F_y$, $0.45F_y$, and $0.30F_u$, respectively. The allowable stresses for bearing are given in the section on Connections.

Load and Resistance Factor Design

The design tensile strength $\phi_t P_n$ for a pin-connected member is given as follows:

Tension on gross area: See Equation 3.10

Tension on effective net area:
$$\phi_t P_n = 0.75[2tb_{eff}F_u] \tag{3.13}$$

Shear on effective area:
$$\phi_{sf} P_n = 0.75[0.6A_{sf}F_u] \tag{3.14}$$

Bearing on projected pin area: See section on Connections

The terms in the above equations are defined as follows:

a = shortest distance from edge of the pin hole to the edge of the member measured in the direction of the force
A_{pb} = projected bearing area $= dt$
A_{sf} = $2t(a + d/2)$
b_{eff} = $2t + 0.63$, but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force
d = pin diameter
t = plate thickness

3.3.4 Threaded Rods

Allowable Stress Design

Threaded rods under tension are treated as bolts subject to tension in allowable stress design. These allowable stresses are given in the section on Connections.
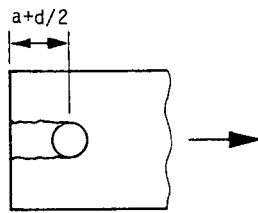
Load and Resistance Factor Design

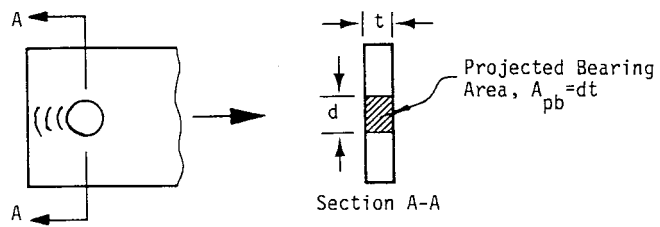Threaded rods designed as tension members shall have a gross area A_b given by

$$A_b \geq \frac{P_u}{\phi 0.75 F_u} \tag{3.15}$$
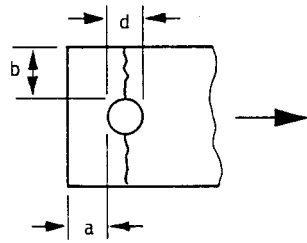
TENSION FRACTURE

LONGITUDINAL SHEAR

BEARING

FIGURE 3.5: Failure modes of pin-connected members.

where

A_b = the gross area of the rod computed using a diameter measured to the outer extremity of the thread
P_u = the factored tensile load
ϕ = the resistance factor given as 0.75
F_u = the specified minimum tensile strength

3.4 Compression Members

Compression members can fail by yielding, inelastic buckling, or elastic buckling depending on the slenderness ratio of the members. Members with low slenderness ratios tend to fail by yielding while members with high slenderness ratios tend to fail by elastic buckling. Most compression members used in construction have intermediate slenderness ratios and so the predominant mode of failure is inelastic buckling. Overall member buckling can occur in one of three different modes: flexural, torsional, and flexural-torsional. Flexural buckling occurs in members with doubly symmetric or doubly antisymmetric cross-sections (e.g., I or Z sections) and in members with singly symmetric sections (e.g., channel, tee, equal-legged angle, double angle sections) when such sections are buckled about an axis that is *perpendicular* to the axis of symmetry. Torsional buckling occurs in members with doubly symmetric sections such as cruciform or built-up shapes with very thin walls. Flexural-torsional buckling occurs in members with singly symmetric cross-sections (e.g., channel, tee, equal-legged angle, double angle sections) when such sections are buckled about the axis of symmetry and in members with unsymmetric cross-sections (e.g., unequal-legged L). Normally, torsional buckling of symmetric shapes is not particularly important in the design of hot-rolled compression members. It either does not govern or its buckling strength does not differ significantly from the corresponding weak axis flexural buckling strengths. However, torsional buckling may become important for open sections with relatively thin component plates. It should be noted that for a given cross-sectional area, a closed section is much stiffer torsionally than an open section. Therefore, if torsional deformation is of concern, a closed section should be used. Regardless of the mode of buckling, the governing effective slenderness ratio (Kl/r) of the compression member preferably should not exceed 200.

In addition to the slenderness ratio and cross-sectional shape, the behavior of compression members is affected by the relative thickness of the component elements that constitute the cross-section. The relative thickness of a component element is quantified by the width-thickness ratio (b/t) of the element. The width-thickness ratios of some selected steel shapes are shown in Figure 3.6. If the width-thickness ratio falls within a limiting value (denoted by the LRFD specification [18] as λ_r) as shown in Table 3.4, the section will not experience local buckling prior to overall buckling of the member. However, if the width-thickness ratio exceeds this limiting width-thickness value, consideration of local buckling in the design of the compression member is required.

To facilitate the design of compression members, column tables for W, tee, double-angle, square/rectangular tubular, and circular pipe sections are available in the AISC Manuals for both allowable stress design [21] and load and resistance factor design [22].

3.4.1 Allowable Stress Design

The computed compressive stress, f_a, in a compression member shall not exceed its allowable value given by

$$F_a = \begin{cases} \dfrac{\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right]f_y}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}}, & \text{if } Kl/r \leq C_c \\ \dfrac{12\pi^2 E}{23(Kl/r)^2}, & \text{if } Kl/r > C_c \end{cases} \qquad (3.16)$$

Flange: $b_f/2t_f$
Web: h/t_w

Flange: $b_f/2t_f$
Web: h/t_w

Flange: b_f/t_f
Web: h/t_w

Flange: b_f/t_f
Stem: d/t_w

Both Legs: b/t

Flange: $(b_f-3t)/t$
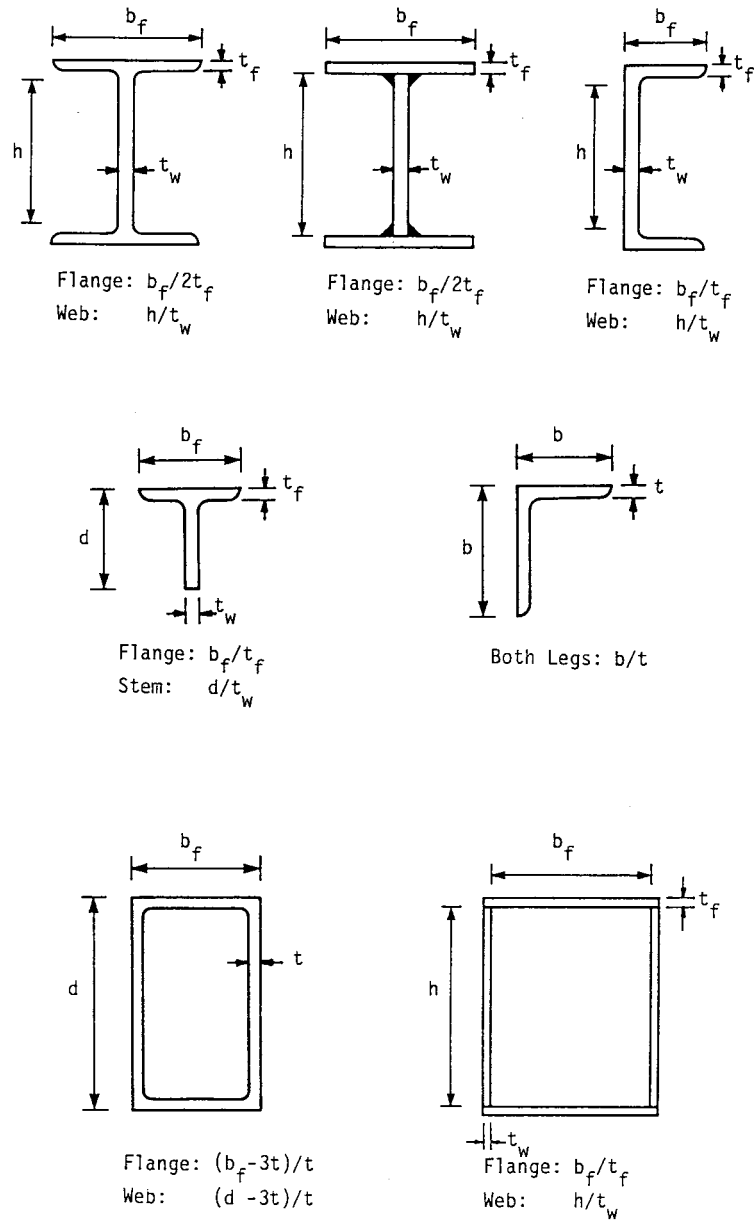Web: $(d-3t)/t$

Flange: b_f/t_f
Web: h/t_w

FIGURE 3.6: Definition of width-thickness ratio of selected cross-sections.

TABLE 3.4 Limiting Width-Thickness Ratios for Compression Elements Under Pure Compression

Component element	Width-thickness ratio	Limiting value, λ_r
Flanges of I-shaped sections; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact; flanges of channels.	b/t	$95/\sqrt{f_y}$
Flanges of square and rectangular box and hollow structural sections of uniform thickness; flange cover plates and diaphragm plates between lines of fasteners or welds.	b/t	$238/\sqrt{f_y}$
Unsupported width of cover plates perforated with a succession of access holes.	b/t	$317/\sqrt{f_y}$
Legs of single angle struts; legs of double angle struts with separators; unstiffened elements (i.e., elements supported along one edge).	b/t	$76/\sqrt{f_y}$
Flanges projecting from built-up members.	b/t	$109/\sqrt{(F_y/k_c^a)}$
Stems of tees.	d/t	$127/\sqrt{F_y}$
All other uniformly compressed elements (i.e., elements supported along two edges).	b/t h/t_w	$253/\sqrt{F_y}$
Circular hollow sections.	D/t $D=$ outside diameter $t=$ wall thickness	$3,300/F_y$

$^a k_c = 4/\sqrt{(h/t_w)}$, and $0.35 \le k_c \le 0.763$ for I-shaped sections, $k_c = 0.763$ for other sections.
$F_y =$ specified minimum yield stress, in ksi.

where Kl/r is the slenderness ratio, K is the effective length factor of the compression member (see Section 3.4.3), l is the unbraced member length, r is the radius of gyration of the cross-section, E is the modulus of elasticity, and $C_c = \sqrt{(2\pi^2 E/F_y)}$ is the slenderness ratio that demarcates between inelastic member buckling from elastic member buckling. Kl/r should be evaluated for both buckling axes and the larger value used in Equation 3.16 to compute F_a.

The first of Equation 3.16 is the allowable stress for inelastic buckling, and the second of Equation 3.16 is the allowable stress for elastic buckling. In ASD, no distinction is made between flexural, torsional, and flexural-torsional buckling.

3.4.2 Load and Resistance Factor Design

Compression members are to be designed so that the design compressive strength $\phi_c P_n$ will exceed the required compressive strength P_u. $\phi_c P_n$ is to be calculated as follows for the different types of overall buckling modes.

Flexural Buckling (with width-thickness ratio $< \lambda_r$):

$$\phi_c P_n = \begin{cases} 0.85 \left[A_g (0.658^{\lambda_c^2}) F_y \right], & \text{if } \lambda_c \le 1.5 \\ 0.85 \left[A_g \left(\frac{0.877}{\lambda_c^2} \right) F_y \right], & \text{if } \lambda_c > 1.5 \end{cases} \tag{3.17}$$

where
$\lambda_c = (KL/r\pi)\sqrt{(F_y/E)}$ is the slenderness parameter
$A_g =$ gross cross-sectional area
$F_y =$ specified minimum yield stress
$E =$ modulus of elasticity
$K =$ effective length factor
$l =$ unbraced member length
$r =$ radius of gyration of the cross-section

The first of Equation 3.17 is the design strength for inelastic buckling and the second of Equation 3.17 is the design strength for elastic buckling. The slenderness parameter $\lambda_c = 1.5$ is therefore the value that demarcates between inelastic and elastic behavior.

Torsional Buckling (with width-thickness ratio $< \lambda_r$):

$\phi_c P_n$ is to be calculated from Equation 3.17, but with λ_c replaced by λ_e given by

$$\lambda_e = \sqrt{(F_y/F_e)} \tag{3.18}$$

where

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ \right] \frac{1}{I_x + I_y} \tag{3.19}$$

in which

C_w = warping constant
G = shear modulus = 11,200 ksi (77,200 MPa)
I_x, I_y = moment of inertia about the major and minor principal axes, respectively
J = torsional constant
K_z = effective length factor for torsional buckling

The warping constant C_w and the torsional constant J are tabulated for various steel shapes in the AISC-LRFD Manual [22]. Equations for calculating approximate values for these constants for some commonly used steel shapes are shown in Table 3.5.

TABLE 3.5   Approximate Equations for C_w and J

Structural shape	Warping constant, C_w	Torsional constant, J
I	$h'^2 I_c I_t / (I_c + I_t)$	$\sum C_i (b_i t_i^3 / 3)$
		where
		b_i = width of component element i
C	$(b' - 3E_o) h'^2 b'^2 t_f / 6 + E_o^2 I_x$	t_i = thickness of component element i
	where	C_i = correction factor for component element i (see values below)
	$E_o = b'^2 t_f / (2b' t_f + h' t_w / 3)$	

		b_i/t_i	C_i
T	$(b_f^3 t_f^3 / 4 + h''^3 t_w^3)/36$	1.00	0.423
	(≈ 0 for small t)	1.20	0.500
		1.50	0.588
		1.75	0.642
L	$(l_1^3 t_1^3 + l_2^3 t_2^3)/36$	2.00	0.687
	(≈ 0 for small t)	2.50	0.747
		3.00	0.789
		4.00	0.843
		5.00	0.873
		6.00	0.894
		8.00	0.921
		10.00	0.936
		∞	1.000

b' = distance measured from toe of flange to center line of web
h' = distance between centerline lines of flanges
h'' = distance from centerline of flange to tip of stem
l_1, l_2 = length of the legs of the angle
t_1, t_2 = thickness of the legs of the angle
b_f = flange width
t_f = average thickness of flange
t_w = thickness of web
I_c = moment of inertia of compression flange taken about the axis of the web
I_t = moment of inertia of tension flange taken about the axis of the web
I_x = moment of inertia of the cross-section taken about the major principal axis

Flexural-Torsional Buckling (with width-thickness ratio $\leq \lambda_r$):

Same as for torsional buckling except F_e is now given by

For singly symmetric sections:

$$F_e = \frac{F_{es} + F_{ez}}{2H}\left[1 - \sqrt{1 - \frac{4F_{es}F_{ez}H}{(F_{es} + F_{ez})^2}}\right] \qquad (3.20)$$

where

F_{es} = F_{ex} if the x-axis is the axis of symmetry of the cross-section, or F_{ey} if the y-axis is the axis of symmetry of the cross-section

F_{ex} = $\pi^2 E/(Kl/r)_x^2$
F_{ey} = $\pi^2 E/(Kl/r)_x^2$
H = $1 - (x_o^2 + y_o^2)/r_o^2$

in which

K_x, K_y = effective length factors for buckling about the x and y axes, respectively
l = unbraced member length
r_x, r_y = radii of gyration about the x and y axes, respectively
x_o, y_o = the shear center coordinates with respect to the centroid Figure 3.7
r_o^2 = $x_o^2 + y_o^2 + r_x^2 + r_y^2$

Numerical values for r_o and H are given for hot-rolled W, channel, tee, and single- and double-angle sections in the AISC-LRFD Manual [22].

For unsymmetric sections:

F_e is to be solved from the cubic equation

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{r_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{r_o}\right)^2 = 0 \quad (3.21)$$

The terms in the above equations are defined the same as in Equation 3.20.

Local Buckling (with width-thickness ratio $\geq \lambda_r$):

Local buckling in a component element of the cross-section is accounted for in design by introducing a reduction factor Q in Equation 3.17 as follows:

$$\phi_c P_n = \begin{cases} 0.85\left[A_g Q\left(0.658^{Q\lambda^2}\right)F_y\right], & \text{if } \lambda\sqrt{Q} \leq 1.5 \\[3mm] 0.85\left[A_g\left(\frac{0.877}{\lambda^2}\right)F_y\right], & \text{if } \lambda\sqrt{Q} > 1.5 \end{cases} \qquad (3.22)$$

where $\lambda = \lambda_c$ for flexural buckling, and $\lambda = \lambda_e$ for flexural-torsional buckling.

The Q factor is given by

$$Q = Q_s Q_a \qquad (3.23)$$

where

Q_s is the reduction factor for unstiffened compression elements of the cross-section (see Table 3.6); and Q_a is the reduction factor for stiffened compression elements of the cross-section (see Table 3.7)

3.4.3 Built-Up Compression Members

Built-up members are members made by bolting and/or welding together two or more standard structural shapes. For a built-up member to be fully effective (i.e., if all component structural shapes are to act as one unit rather than as individual units), the following conditions must be satisfied:
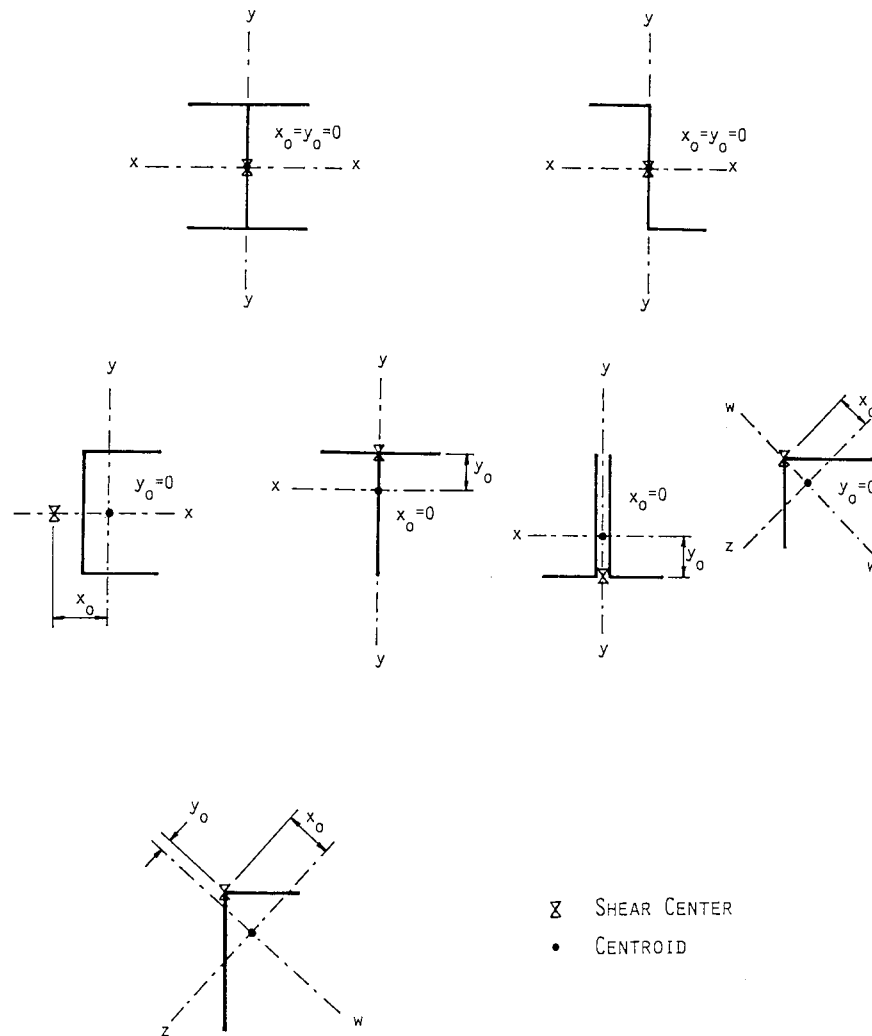
FIGURE 3.7: Location of shear center for selected cross-sections.

1. The ends of the built-up member must be prevented from slippage during buckling.
2. Adequate fasteners must be provided along the length of the member.
3. The fasteners must be able to provide sufficient gripping force on all the component shapes being connected.

Condition 1 is satisfied if all component shapes in contact at the ends of the member are connected by a weld having a length not less than the maximum width of the member or by fully tightened bolts spaced longitudinally not more than four diameters apart for a distance equal to 1-1/2 times the maximum width of the member.

Condition 2 is satisfied if continuous welds are used throughout the length of the built-up compression member.

Condition 3 is satisfied if either welds or fully tightened bolts are used as the fasteners.

While condition 1 is mandatory, conditions 2 and 3 can be violated in design. If condition 2 or 3 is violated, the built-up member is not fully effective and slight slippage among component shapes

TABLE 3.6   Formulas for Q_s

Structural element	Range of b/t	Q_s
Single angles	$76.0/\sqrt{F_y} < b/t < 155/\sqrt{F_y}$	$1.340 - 0.00447(b/t)\sqrt{f_y}$
	$b/t \geq 155/\sqrt{f_y}$	$15,500/[F_y(b/t)^2]$
Flanges, angles, and plates projecting from columns or other compression members	$95.0/\sqrt{F_y} < b/t < 176/\sqrt{f_y}$	$1.415 - 0.00437(b/t)\sqrt{f_y}$
	$b/t \geq 176/\sqrt{F_y}$	$20,000/[F_y(b/t)^2]$
Flanges, angles, and plates projecting from built-up columns or other compression members	$109/\sqrt{(F_y/k_c^a)} < b/t < 200/\sqrt{(F_y/k_c)}$	$1.415 - 0.00381(b/t)\sqrt{(F_y/k_c)}$
	$b/t \geq 200/\sqrt{(F_y/k_c)}$	$26,200kc/[F_y(b/t)^2]$
Stems of tees	$127/\sqrt{F_y} < b/t < 176/\sqrt{F_y}$	$1.908 - 0.00715(b/t)\sqrt{F_y}$
	$b/t \geq 176/\sqrt{f_y}$	$20,000/[F_y(b/t)^2]$

[a] see footnote a in Table 3.4
F_y = specified minimum yield stress, in ksi
b = width of the component element
t = thickness of the component element

TABLE 3.7   Formula for Q_a

$$Q_s = \frac{\text{effective area}}{\text{actual area}}$$

The effective area is equal to the summation of the effective areas of the stiffened elements of the cross-section. The effective area of a stiffened element is equal to the product of its thickness t and its effective width b_e given by:

For flanges of square and rectangular sections of uniform thickness: when $b/t \geq \frac{238^a}{\sqrt{f}}$

$$b_e = \frac{326t}{\sqrt{f}}\left[1 - \frac{64.9}{(b/t)\sqrt{f}}\right] \leq b$$

For other uniformly compressed elements: when $b/t \geq \frac{253^a}{\sqrt{f}}$

$$b_e = \frac{326t}{\sqrt{f}}\left[1 - \frac{57.2}{(b/t)\sqrt{f}}\right] \leq b$$

where
b = actual width of the stiffened element
f = computed elastic compressive stress in the stiffened elements, in ksi

[a] b_e = b otherwise.

may occur. To account for the decrease in capacity due to slippage, a modified slenderness ratio is used for the computation of the design compressive strength when buckling of the built-up member is about an axis *coincide* or *parallel* to at least one plane of contact for the component shapes. The modified slenderness ratio $(KL/r)_m$ is given as follows:

If condition 2 is violated:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \frac{0.82\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2}$$

(3.24)

If conditions 2 and 3 are violated:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \tag{3.25}$$

In the above equations, $(KL/r)_o = (KL/r)_x$ if the buckling axis is the x-axis and at least one plane of contact between component shapes is parallel to that axis; $(KL/r)_o = (KL/r)_y$ if the buckling axis is the y axis and at least one plane of contact is parallel to that axis. a is the longitudinal spacing of the fasteners, r_i is the minimum radius of gyration of any component element of the built-up cross-section, r_{ib} is the radius of gyration of an individual component relative to its centroidal axis parallel to the axis of buckling of the member, h is the distance between centroids of component elements measured perpendicularly to the buckling axis of the built-up member.

No modification to (KL/r) is necessary if the buckling axis is perpendicular to the planes of contact of the component shapes. Modifications to both $(KL/r)_x$ and $(KL/r)_y$ are required if the built-up member is so constructed that planes of contact exist in both the x and y directions of the cross-section.

Once the modified slenderness ratio is computed, it is to be used in the appropriate equation to calculate F_a in allowable stress design, or $\phi_c P_n$ in load and resistance factor design.

An additional requirement for the design of built-up members is that the effective slenderness ratio, Ka/r_i, of each component shape, where K is the effective length factor of the component shape between adjacent fasteners, does not exceed 3/4 of the governing slenderness ratio of the built-up member. This provision is provided to prevent component shape buckling between adjacent fasteners from occurring prior to overall buckling of the built-up member.

EXAMPLE 3.2:

Using LRFD, determine the size of a pair of cover plates to be bolted, using snug-tight bolts, to the flanges of a W24x229 section as shown in Figure 3.8 so that its design strength, $\phi_c P_n$, will be increased by 15%. Also, determine the spacing of the bolts in the longitudinal direction of the built-up column.
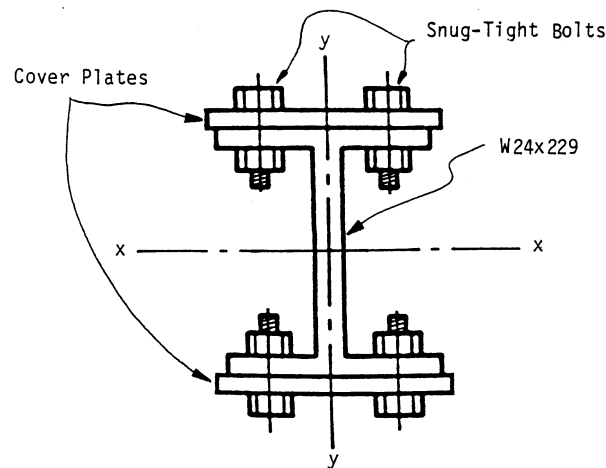

FIGURE 3.8: Design of cover plates for a compression member.

The effective lengths of the section about the major $(KL)_x$ and minor $(KL)_y$ axes are both equal to 20 ft. A36 steel is to be used.

Determine design strength for the W24x229 section:

Since $(KL)_x = (KL)_y$ and $r_x > r_y$, $(KL/r)_y$ will be greater than $(KL/r)_x$ and the design strength will be controlled by flexural buckling about the minor axis. Using section properties, $r_y = 3.11$ in. and $A = 67.2$ in.^{2}, obtained from the AISC-LRFD Manual [22], the slenderness parameter λ_c about the minor axis can be calculated as follows:

$$(\lambda_c)_y = \frac{1}{\pi} \left(\frac{KL}{r} \right)_y \sqrt{\frac{F_y}{E}} = \frac{1}{3.142} \left(\frac{20 \times 12}{3.11} \right) \sqrt{\frac{36}{29,000}} = 0.865$$

Substituting $\lambda_c = 0.865$ into Equation 3.17, the design strength of the section is

$$\phi_c P_n = 0.85 \left[67.2 \left(0.658^{0.865^2} \right) 36 \right] = 1503 \text{ kips}$$

Alternatively, the above value of $\phi_c P_n$ can be obtained directly from the column tables contained in the AISC-LRFD Manual.

Determine design strength for the built-up section:

The built-up section is expected to possess a design strength which is 15% in excess of the design strength of the W24x229 section, so

$$(\phi_c P_n)_{req'd} = (1.15)(1503) = 1728 \text{ kips}$$

Determine size of the cover plates:

After cover plates are added, the resulting section is still doubly symmetric. Therefore, the overall failure mode is still flexural buckling. For flexural buckling about the minor axis (y-y), no modification to (KL/r) is required because the buckling axis is perpendicular to the plane of contact of the component shapes and no relative movement between the adjoining parts is expected. However, for flexural buckling about the major (x-x) axis, modification to (KL/r) is required because the buckling axis is parallel to the plane of contact of the adjoining structural shapes and slippage between the component pieces will occur. We shall design the cover plates assuming flexural buckling about the minor axis will control and check for flexural buckling about the major axis later.

A W24x229 section has a flange width of 13.11 in.; so, as a trial, use cover plates with widths of 13 in. as shown in Figure 3.8a. Denoting t as the thickness of the plates, we have

$$(r_y)_{\text{built-up}} = \sqrt{\frac{(I_y)_{\text{W-shape}} + (I_y)_{\text{plates}}}{A_{\text{W-shape}} + A_{\text{plates}}}} = \sqrt{\frac{651 + 183.1t}{67.2 + 26t}}$$

and

$$(\lambda_c)_{y,\text{built-up}} = \frac{1}{\pi} \left(\frac{KL}{r} \right)_{y,\text{built-up}} \sqrt{\frac{F_y}{E}} = 2.69 \sqrt{\frac{67.2 + 26t}{651 + 183.1t}}$$

Assuming $(\lambda)_{y,\text{built-up}}$ is less than 1.5, one can substitute the above expression for λ_c in Equation 3.17. With $\phi_c P_n$ equals 1728, we can solve for t. The result is $t = 1/2$ in. Backsubstituting $t = 1/2$ into the above expression, we obtain $(\lambda)_{c,\text{built-up}} = 0.884$ which is indeed <1.5. So, try 13"x1/2" cover plates.

Check for local buckling:
For the I-section:

$$\text{Flange:} \quad \left[\frac{b_f}{2t_f} = 3.8\right] \quad < \quad \left[\frac{95}{\sqrt{F_y}} = 15.8\right]$$

$$\text{Web:} \quad \left[\frac{h_c}{t_w} = 22.5\right] \quad < \quad \left[\frac{253}{\sqrt{F_y}} = 42.2\right]$$

For the cover plates, if 3/4-in. diameter bolts are used and assuming an edge distance of 1-1/4 in., the width of the plate between fasteners will be $13-2.5 = 10.5$ in. Therefore, we have

$$\left[\frac{b}{t} = \frac{10.5}{1/2} = 21\right] < \left[\frac{238}{\sqrt{F_y}} = \frac{238}{\sqrt{36}} = 39.7\right]$$

Since the width-thickness ratios of all component shapes do not exceed the limiting width-thickness ratio for local buckling, local buckling is not a concern.

Check for flexural buckling about the major (x-x) axis:
Since the built-up section is doubly symmetric, the governing buckling mode will be flexural buckling regardless of the axes. Flexural buckling will occur about the major axis if the modified slenderness ratio $(KL/r)_m$ about the major axis exceeds $(KL/r)_y$. Therefore, as long as $(KL/r)_m$ is less than $(KL/r)_y$, buckling will occur about the minor axis and flexural buckling about the major axis will not be controlled. In order to arrive at an optimal design, we shall determine the longitudinal fastener spacing, a, such that the modified slenderness ratio $(KL/r)_m$ about the major axis will be equal to $(KL/r)_y$. That is, we shall solve for a from the equation

$$\left[\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_x^2 + \left(\frac{a}{r_i}\right)^2}\right] = \left[\left(\frac{KL}{r}\right)_y = 78.9\right]$$

In the above equation, $(KL/r)_x$ is the slenderness ratio about the major axis of the built-up section, r_i is the least radius of gyration of the component shapes, which in this case is the cover plate. Substituting $(KL/r)_x = 21.56$, $r_i = r_{\text{cover plate}} = \sqrt{(I/A)_{\text{cover plate}}} = \sqrt{[(1/2)^2/112]} = 0.144$ into the above equation, we obtain $a = 10.9$ in. Since $(KL) = 20$ ft, we shall use $a = 10$ in. for the longitudinal spacing of the fasteners.

Check for component shape buckling between adjacent fasteners:

$$\left[\frac{Ka}{r_i} = \frac{1 \times 10}{0.144} = 69.44\right] > \left[\frac{3}{4}\left(\frac{KL}{r}\right)_y = \frac{3}{4}(78.9) = 59.2\right]$$

Since the component shape buckling criterion is violated, we need to decrease the longitudinal spacing from 10 in. to 8 in.

Use 13"x1/2" cover plates bolted to the flanges of the W24x229 section by 3/4-in. diameter fully tightened bolts spaced 8 in. longitudinally.

3.5   Flexural Members

Depending on the width-thickness ratios of the component elements, steel sections used for flexural members are classified as compact, noncompact, and slender element sections. Compact sections

are sections that can develop the cross-section plastic moment (M_p) under flexure and sustain that moment through a large hinge rotation without fracture. Noncompact sections are sections that either cannot develop the cross-section full plastic strength or cannot sustain a large hinge rotation at M_p, probably due to local buckling of the flanges or web. Slender element sections are sections that fail by local buckling of component elements long before M_p is reached. A section is considered compact if all its component elements have width-thickness ratios less than a limiting value (denoted as λ_p in LRFD). A section is considered noncompact if one or more of its component elements have width-thickness ratios that fall in between λ_p and λ_r. A section is considered to be a slender element if one or more of its component elements have width-thickness ratios that exceed λ_r. Expressions for λ_p and λ_r are given in the Table 3.8

In addition to the compactness of the steel section, another important consideration for beam design is the lateral unsupported (unbraced) length of the member. For beams bent about their strong axes, the failure modes, or limit states, vary depending on the number and spacing of lateral supports provided to brace the compression flange of the beam. The compression flange of a beam behaves somewhat like a compression member. It buckles if adequate lateral supports are not provided in a phenomenon called lateral torsional buckling. *Lateral torsional buckling* may or may not be accompanied by yielding, depending on the lateral unsupported length of the beam. Thus, lateral torsional buckling can be inelastic or elastic. If the lateral unsupported length is large, the limit state is elastic lateral torsional buckling. If the lateral unsupported length is smaller, the limit state is inelastic lateral torsional buckling. For compact section beams with adequate lateral supports, the limit state is full yielding of the cross-section (i.e., plastic hinge formation). For noncompact section beams with adequate lateral supports, the limit state is flange or web local buckling.

For beams bent about their weak axes, lateral torsional buckling will not occur and so the lateral unsupported length has no bearing on the design. The limit states for such beams will be formation of a plastic hinge if the section is compact. The limit state will be flange or web local buckling if the section is noncompact.

Beams subjected to high shear must be checked for possible web shear failure. Depending on the width-thickness ratio of the web, failure by shear yielding or web shear buckling may occur. Short, deep beams with thin webs are particularly susceptible to web shear failure. If web shear is of concern, the use of thicker webs or web reinforcements such as stiffeners is required.

Beams subjected to concentrated loads applied in the plane of the web must be checked for a variety of possible flange and web failures. Failure modes associated with concentrated loads include local flange bending (for tensile concentrated load), local web yielding (for compressive concentrated load), web crippling (for compressive load), sidesway web buckling (for compressive load), and compression buckling of the web (for a compressive load pair). If one or more of these conditions is critical, transverse stiffeners extending at least one-half the beam depth (use full depth for compressive buckling of the web) must be provided adjacent to the concentrated loads.

Long beams can have deflections that may be too excessive, leading to problems in serviceability. If deflection is excessive, the use of intermediate supports or beams with higher flexural rigidity is required.

The design of flexural members should satisfy the following criteria: (1) flexural strength criterion, (2) shear strength criterion, (3) criteria for concentrated loads, and (4) deflection criterion. To facilitate beam design, a number of beam tables and charts are given in the AISC Manuals [21, 22] for both allowable stress and load and resistance factor design.

TABLE 3.8 λ_p and λ_r for Members Under Flexural Compression

Component element	Width-thickness ratio[a]	λ_p	λ_r
Flanges of I-shaped rolled beams and channels	b/t	$65/\sqrt{F_y}$	$141/\sqrt{(F_y - 10)}$[b]
Flanges of I-shaped hybrid or welded beams	b/t	$65/\sqrt{F_{yf}}$ (non-seismic) $52/\sqrt{F_{yf}}$ (seismic) F_{yf} = yield stress of flange	$162/\sqrt{(F_{yf} - 16.5)/k_c}$[c] F_{yw} = yield stress of web
Flanges of square and rectangular box and hollow structural sections of uniform thickness; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$190/\sqrt{F_y}$	$238/\sqrt{F_y}$
Unsupported width of cover plates perforated with a succession of access holes	b/t	NA	$317/\sqrt{F_y}$
Legs of single angle struts; legs of double angle struts with separators; unstiffened elements	b/t	NA	$76/\sqrt{F_y}$
Stems of tees	d/t	NA	$127/\sqrt{F_y}$
Webs in flexural compression	h_c/t_w	$640/\sqrt{F_y}$ (non-seismic) $520/\sqrt{F_y}$ (seismic)	$970/\sqrt{F_y}$[d]
Webs in combined flexural and axial compression	h_c/t_w	For $P_u/\phi_b P_y \le 0.125$: $640(1 - 2.75 P_u/\phi_b P_y)/\sqrt{F_y}$ (non-seismic) $520(1 - 1.54 P_u/\phi_b P_y)/\sqrt{F_y}$ (seismic) For $P_u/\phi_b P_y > 0.125$: $191(2.33 - P_u/\phi_b P_y)/\sqrt{F_y}$ $\ge 253/\sqrt{F_y}$ $\phi_b = 0.90$ P_u = factored axial force; $P_y = A_g F_y$.	$970/\sqrt{F_y}$[d]
Circular hollow sections	D/t D = outside diameter; t = wall thickness	$2,070/F_y$ $1,300/F_y$ for plastic design	$8,970/F_y$

[a] See Figure 3.6 for definition of b, h_c, and t

[b] For ASD, this limit is $95/\sqrt{F_y}$

[c] For ASD, this limit is $95/\sqrt{(F_{yf}/k_c)}$, where $k_c = 4.05/(h/t)^{0.46}$ if $h/t > 70$, otherwise $k_c = 1.0$

[d] For ASD, this limit is $760/\sqrt{F_b}$

Note: All stresses have units of ksi.

3.5.1 Allowable Stress Design

Flexural Strength Criterion

The computed flexural stress, f_b, shall not exceed the allowable flexural stress, F_b, given as follows (in all equations, the minimum specified yield stress, F_y, cannot exceed 65 ksi):

Compact-Section Members Bent About Their Major Axes

For $L_b \leq L_c$,

$$F_b = 0.66F_y \tag{3.26}$$

where

L_c = smaller of $\{76b_f/\sqrt{F_y}, 20000/(d/A_f)F_y\}$, for I and channel shapes

      = $[1950 + 1200(M_1/M_2)](b/F_y) \geq 1200(b/F_y)$, for box sections, rectangular and circular tubes

in which

b_f = flange width, in.

d = overall depth of section, ksi

A_f = area of compression flange, in.^{2}

b = width of cross-section, in.

M_1/M_2 = ratio of the smaller to larger moment at the ends of the unbraced length of the beam. M_1/M_2 is positive for reverse curvature bending and negative for single curvature bending.

For the above sections to be considered compact, in addition to having the width-thickness ratios of their component elements falling within the limiting value of λ_p shown in Table 3.8, the flanges of the sections must be continuously connected to the webs. For box-shaped sections, the following requirements must also be satisfied: the depth-to-width ratio should not exceed six, and the flange-to-web thickness ratio should exceed two.

For $L_b > L_c$, the allowable flexural stress in tension is given by

$$F_b = 0.60F_y \tag{3.27}$$

and the allowable flexural stress in compression is given by the larger value calculated from Equation 3.28 and Equation 3.29. Equation 3.28 normally controls for deep, thin-flanged sections where warping restraint torsional resistance dominates, and Equation 3.29 normally controls for shallow, thick-flanged sections where St. Venant torsional resistance dominates.

$$F_b = \begin{cases} \left[\dfrac{2}{3} - \dfrac{F_y(l/r_T)^2}{1530 \times 10^3 C_b}\right] F_y \leq 0.60F_y, & \text{if } \sqrt{\dfrac{102,000C_b}{F_y}} \leq \dfrac{l}{r_T} < \sqrt{\dfrac{510,000C_b}{F_y}} \\[4mm] \dfrac{170,000C_b}{(l/r_T)^2} \leq 0.60F_y, & \text{if } \dfrac{l}{r_T} \geq \sqrt{\dfrac{510,000C_b}{F_y}} \end{cases} \tag{3.28}$$

$$F_b = \frac{12,000C_b}{ld/A_f} \leq 0.60F_y \tag{3.29}$$

where

l = distance between cross-sections braced against twist or lateral displacement of the compression flange, in.

r_T = radius of gyration of a section comprising the compression flange plus 1/3 of the compression web area, taken about an axis in the plane of the web, in.

A_f = compression flange area, in.^{2}

C_b = $12.5M_{\max}/(2.5M_{\max} + 3M_A + 4M_B + 3M_C)$

$M_{\max}, M_A, M_B, M_C$ = maximum moment, quarter-point moment, midpoint moment, and three-quarter point moment along the unbraced length of the member, respectively.

For simplicity in design, C_b can conservatively be taken as unity.

It should be cautioned that Equations 3.28 and 3.29 are applicable only to I and channel shapes with an axis of symmetry in, and loaded in the plane of the web. In addition, Equation 3.29 is applicable only if the compression flange is solid and approximately rectangular in shape, and its area is not less than the tension flange.

Compact Section Members Bent About Their Minor Axes

Since lateral torsional buckling will not occur for bending about the minor axes, regardless of the value of L_b, the allowable flexural stress is

$$F_b = 0.75F_y \tag{3.30}$$

Noncompact Section Members Bent About Their Major Axes

For $L_b \le L_c$,

$$F_b = 0.60F_y \tag{3.31}$$

where L_c is defined as for Equation 3.26.

For $L_b > L_c$, F_b is given in Equation 3.27, 3.28, or 3.29.

Noncompact Section Members Bent About Their Minor Axes

Regardless of the value of L_b,

$$F_b = 0.60F_y \tag{3.32}$$

Slender Element Sections

Refer to the section on Plate Girders.

Shear Strength Criterion

For practically all structural shapes commonly used in constructions, the shear resistance from the flanges is small compared to the webs. As a result, the shear resistance for flexural members is normally determined on the basis of the webs only. The amount of web shear resistance is dependent on the width-thickness ratio h/t_w of the webs. If h/t_w is small, the failure mode is web yielding. If h/t_w is large, the failure mode is web buckling. To avoid web shear failure, the computed shear stress, f_v, shall not exceed the allowable shear stress, F_v, given by

$$F_v = \begin{cases} 0.40F_y, & \text{if } \frac{h}{t_w} \le \frac{380}{\sqrt{F_y}} \\ \frac{C_v}{2.89}F_y \le 0.40F_y, & \text{if } \frac{h}{t_w} > \frac{380}{\sqrt{F_y}} \end{cases} \tag{3.33}$$

where

C_v = $45,000k_v/F_y(h/t_w)^2$, if $C_v \le 0.8$
   = $190\sqrt{(k_v/F_y)}/(h/t_w)$, if $C_v > 0.8$
k_v = $4.00 + 5.34/(a/h)^2$, if $a/h \le 1.0$
   = $5.34 + 4.00/(a/h)^2$, if $a/h > 1.0$
t_w = web thickness, in.
a = clear distance between transverse stiffeners, in.
h = clear distance between flanges at section under investigation, in.

Criteria for Concentrated Loads

Local Flange Bending

If the concentrated force that acts on the beam flange is tensile, the beam flange may experience excessive bending, leading to failure by fracture. To preclude this type of failure, transverse stiffeners are to be provided opposite the tension flange unless the length of the load when measured across the beam flange is less than 0.15 times the flange width, or if the flange thickness, t_f, exceeds

$$0.4\sqrt{\frac{P_{bf}}{F_y}} \tag{3.34}$$

where

P_{bf} = computed tensile force multiplied by 5/3 if the force is due to live and dead loads only, or by 4/3 if the force is due to live and dead loads in conjunction with wind or earthquake loads, kips.

F_y = specified minimum yield stress, ksi.

Local Web Yielding

To prevent local web yielding, the concentrated compressive force, R, should not exceed $0.66R_n$, where R_n is the web yielding resistance given in Equation 3.52 or Equation 3.53, whichever applies.

Web Crippling

To prevent web crippling, the concentrated compressive force, R, should not exceed $0.50R_n$, where R_n is the web crippling resistance given in Equation 3.54, Equation 3.55, or Equation 3.56, whichever applies.

Sidesway Web Buckling

To prevent sidesway web buckling, the concentrated compressive force, R, should not exceed R_n, where R_n is the sidesway web buckling resistance given in Equation 3.57 or Equation 3.58, whichever applies, except the term $C_r t_w^3 t_f / h^2$ is replaced by $6{,}800 t_w^3 / h$.

Compression Buckling of the Web

When the web is subjected to a pair of concentrated forces acting on both flanges, buckling of the web may occur if the web depth clear of fillet, d_c, is greater than

$$\frac{4100 t_w^3 \sqrt{F_y}}{P_{bf}} \tag{3.35}$$

where t_w is the web thickness, F_y is the minimum specified yield stress, and P_{bf} is as defined in Equation 3.34.

Deflection Criterion

Deflection is a serviceability consideration. Since most beams are fabricated with a camber which somewhat offsets the dead load deflection, consideration is often given to deflection due to live load only. For beams supporting plastered ceilings, the service live load deflection preferably should not exceed $L/360$ where L is the beam span. A larger deflection limit can be used if due considerations are given to ensure the proper functioning of the structure.

EXAMPLE 3.3:

Using ASD, determine the amount of increase in flexural capacity of a W24x55 section bent about its major axis if two 7"x1/2" (178mmx13mm) cover plates are bolted to its flanges as shown in
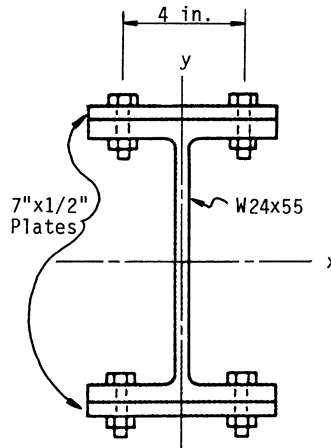
FIGURE 3.9: Cover-plated beam section.

Figure 3.9. The beam is laterally supported at every 5-ft (1.52-m) interval. Use A36 steel. Specify the type, diameter, and longitudinal spacing of the bolts used if the maximum shear to be resisted by the cross-section is 100 kips (445 kN).

Section properties:
A W24x55 section has the following section properties:
b_f =7.005 in. t_f =0.505 in. d =23.57 in. t_w =0.395 in. I_x =1350 in.[4] S_x =114 in.[3]
Check compactness:
Refer to Table 3.8, and assuming that the transverse distance between the two bolt lines is 4 in., we have

$$\text{Beam flanges} \quad \left[\frac{b_f}{2t_f} = 6.94"\right] \quad < \quad \left[\frac{65}{\sqrt{F_y}} = 10.8"\right]$$

$$\text{Beam web} \quad \left[\frac{d}{t_w} = 59.7"\right] \quad < \quad \left[\frac{640}{\sqrt{F_y}} = 107"\right]$$

$$\text{Cover plates} \quad \left[\frac{4}{1/2} = 8"\right] \quad < \quad \left[\frac{190}{\sqrt{F_y}} = 31.7"\right]$$

Therefore, the section is compact.

Determine the allowable flexural stress, F_b:
Since the section is compact and the lateral unbraced length, $L_b = 60$ in., is less than $L_c = 83.4$ in., the allowable bending stress from Equation 3.26 is $0.66F_y$ =24 ksi.

Determine section modulus of the beam with cover plates:

$$
\begin{aligned}
S_{x,\text{combination section}} &= \frac{I_{x,\text{combination section}}}{c} \\
&= \frac{1350 + 2\left[\left(\frac{1}{12}\right)(7)(1/2)^3 + (7)(1/2)(12.035)^2\right]}{(11.785 + 1/2)} = 192 \ \text{in.}^3
\end{aligned}
$$

Determine flexural capacity of the beam with cover plates:

$$M_{x,\text{combination section}} = S_{x,\text{combination section}} F_b = (192)(24) = 4608 \ \text{k-in.}$$

Since the flexural capacity of the beam without cover plates is

$$M_x = S_x F_b = (114)(24) = 2736 \text{ k-in.}$$

the increase in flexural capacity is 68.4%.

Determine diameter and longitudinal spacing of bolts:

From *Mechanics of Materials,* the relationship between the shear flow, q, the number of bolts per shear plane, n, the allowable bolt shear stress, F_v, the cross-sectional bolt area, A_b, and the longitudinal bolt spacing, s, at the interface of two component elements of a combination section is given by

$$\frac{n F_v A_b}{s} = q$$

Substituting $n = 2$, $q = VQ/I = (100)[(7)(1/2)(12.035)]/2364 = 1.78$ k/in. into the above equation, we have

$$\frac{F_v A_b}{s} = 0.9 \text{ k/in.}$$

If 1/2" diameter A325N bolts are used, we have $A_b = \pi(1/2)^2/4 = 0.196$ in.^{2}, and $F_v = 21$ ksi (from Table 3.12), from which s can be solved from the above equation to be 4.57 in. However, for ease of installation, use $s = 4.5$ in.

In calculating the section properties of the combination section, no deduction is made for the bolt holes in the beam flanges nor the cover plates. This is allowed provided that the following condition is satisfied:

$$0.5 F_u A_{fn} \geq 0.6 F_y A_{fg}$$

where F_y and F_u are the minimum specified yield strength and tensile strength, respectively. A_{fn} is the net flange area and A_{fg} is the gross flange area. For this problem

Beam flanges

$$\left[0.5 F_u A_{fn} = 0.5(58)(7.005 - 2 \times 1/2)(0.505) = 87.9 \text{ kips} \right]$$

$$> \left[0.6 F_y A_{fg} = 0.6(36)(7.005)(0.505) = 76.4 \text{ kips} \right]$$

Cover Plates

$$\left[0.5 F_u A_{fn} = 0.5(58)(7 - 2 \times 1/2)(1/2) = 87 \text{ kips} \right]$$

$$> \left[0.6 F_y A_{fg} = 0.6(36)(7)(1/2) = 75.6 \text{ kips} \right]$$

so the use of the gross cross-sectional area to compute section properties is justified. In the event that the condition is violated, cross-sectional properties should be evaluated using an effective tension flange area A_{fe} given by

$$A_{fe} = \frac{5}{6} \frac{F_u}{F_y} A_{fn}$$

Use 1/2" diameter A325N bolts spaced 4.5" apart longitudinally in two lines 4" apart to connect the cover plates to the beam flanges.

3.5.2 Load and Resistance Factor Design

Flexural Strength Criterion

Flexural members must be designed to satisfy the flexural strength criterion of

$$\phi_b M_n \geq M_u \tag{3.36}$$

where $\phi_b M_n$ is the design flexural strength and M_u is the required strength. The design flexural strength is determined as follows:

Compact Section Members Bent About Their Major Axes

For $L_b \leq L_p$, (Plastic hinge formation)

$$\phi_b M_n = 0.90 M_p \tag{3.37}$$

For $L_p < L_b \leq L_r$, (Inelastic lateral torsional buckling)

$$\phi_b M_n = 0.90 C_b \left[M_p - (M_p - M_r) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq 0.90 M_p \tag{3.38}$$

For $L_b > L_r$, (Elastic lateral torsional buckling)

For I-shaped members and channels:

$$\phi_b M_n = 0.90 C_b \left[\frac{\pi}{L_b} \sqrt{E I_y G J + \left(\frac{\pi E}{L_b} \right)^2 I_y C_w} \right] \leq 0.90 M_p \tag{3.39}$$

For solid rectangular bars and symmetric box sections:

$$\phi_b M_n = 0.90 C_b \frac{57,000 \sqrt{J A}}{L_b / r_y} \leq 0.90 M_p \tag{3.40}$$

The variables used in the above equations are defined in the following.

L_b =   lateral unsupported length of the member

L_p, L_r =   limiting lateral unsupported lengths given in the following table

Structural shape	L_p	L_r
I-shaped sections, chanels	$300 r_y / \sqrt{F_{yf}}$	$[r_y X_1 / F_L] \left\{ \sqrt{\left[1 + \sqrt{\left(1 + X_2 F_L^2 \right)} \right]} \right\}$
	where r_y = radius of gyration about minor axis, in. F_{yf} = flange yield stress, ksi	where r_y = radius of gyration about minor axis, in. $X_1 = (\pi / S_x)\sqrt{(EGJA/2)}$ $X_2 = (4 C_w / I_y)(S_x / GJ)^2$ F_L = smaller of $(F_{yf} - F_r)$ or F_{yw} F_{yf} = flange yield stress, ksi F_{yw} = web yield stress, ksi F_r = 10 ksi for rolled shapes, 16.5 ksi for welded shapes S_x = elastic section modulus about the major axis, in.^{3} (use S_{xc}, the elastic section modulus about the major axis with respect to the compression flange if the compression flange is larger than the tension flange) I_y = moment of inertia about the minor axis, in.^{4} J = torsional constant, in.^{4} C_w = warping constant, in.^{6} E = modulus of elasticity, ksi G = shear modulus, ksi
Solid rectangular bars, symmetric box sections	$\left[3,750 r_y \sqrt{(JA)} \right] / M_p$	$\left[57,000 r_y \sqrt{(JA)} \right] / M_r$
	where r_y = radius of gyration about minor axis, in. J = torsional constant, in.^{4} A = cross-sectional area, in.^{2} M_p = plastic moment capacity $= F_y Z_x$ F_y = yield stress, ksi Z_x = plastic section modulus about the major axis, in.^{3}	where r_y = radius of gyration about minor axis, in. J = torsional constant, in.^{4} A = cross-sectional area, in.^{2} $M_r = F_y S_x$ for solid rectangular bar, $F_{yf} S_{eff}$ for box sections F_y = yield stress, ksi F_{yf} = flange yield stress, ksi S_x = plastic section modulus about the major axis, in.^{3}

Note: L_p given in this table are valid only if the bending coefficient C_b is equal to unity. If $C_b > 1$, the value of L_p can be increased. However, using the L_p expressions given above for $C_b > 1$ will give a conservative value for the flexural design strength.

and

$M_p = F_y Z_x$

$M_r = F_L S_x$ for I-shaped sections and channels, $F_y S_x$ for solid rectangular bars, $F_{yf} S_{eff}$ for box sections

F_L = smaller of $(F_{yf} - F_r)$ or F_{yw}

F_{yf} = flange yield stress, ksi

F_{yw} = web yield stress

F_r = 10 ksi for rolled sections, 16.5 ksi for welded sections

F_y = specified minimum yield stress

S_x = elastic section modulus about the major axis

S_{eff} = effective section modular, calculated using effective width b_e, in Table 3.7

Z_x = plastic section modulus about the major axis

I_y = moment of inertia about the minor axis

J = torsional constant

C_w = warping constant

E = modulus of elasticity

G = shear modulus

$C_b = 12.5 M_{\max} / (2.5 M_{\max} + 3 M_A + 4 M_B + 3 M_C)$

$$M_{\max}, M_A, M_B, M_C \quad = \quad \text{maximum moment, quarter-point moment, midpoint moment, and}$$
three-quarter point moment along the unbraced length of the member, respectively.

C_b is a factor that accounts for the effect of moment gradient on the lateral torsional buckling strength of the beam. Lateral torsional buckling strength increases for a steep moment gradient. The worst loading case as far as lateral torsional buckling is concerned is when the beam is subjected to a uniform moment resulting in single curvature bending. For this case $C_b = 1$. Therefore, the use of $C_b = 1$ is conservative for the design of beams.

Compact Section Members Bent About Their Minor Axes
Regardless of L_b, the limit state will be a plastic hinge formation

$$\phi_b M_n = 0.90 M_{py} = 0.90 F_y Z_y \tag{3.41}$$

Noncompact Section Members Bent About Their Major Axes
For $L_b \le L'_p$, (Flange or web local buckling)

$$\phi_b M_n = \phi_b M'_n = 0.90 \left[M_p - (M_p - M_r) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right] \tag{3.42}$$

where

$$L'_p = L_p + (L_r - L_p) \left(\frac{M_p - M'_n}{M_p - M_r} \right) \tag{3.43}$$

L_p, L_r, M_p, M_r are defined as before for compact section members, and

For flange local buckling:
$\lambda \quad = \quad b_f/2t_f$ for I-shaped members, b_f/t_f for channels
$\lambda_p \quad = \quad 65/\sqrt{F_y}$
$\lambda_r \quad = \quad 141/\sqrt{(F_y - 10)}$

For web local buckling:
$\lambda \quad = \quad h_c/t_w$
$\lambda_p \quad = \quad 640/\sqrt{F_y}$
$\lambda_r \quad = \quad 970/\sqrt{F_y}$

in which
$b_f \quad = \quad$ flange width
$t_f \quad = \quad$ flange thickness
$h_c \quad = \quad$ twice the distance from the neutral axis to the inside face of the compression flange less the fillet or corner radius
$t_w \quad = \quad$ web thickness

For $L'_p < L_b \le L_r$, (Inelastic lateral torsional buckling), $\phi_b M_n$ is given by Equation 3.38 except that the limit $0.90 M_p$ is to be replaced by the limit $0.90 M'_n$.

For $L_b > L_r$, (Elastic lateral torsional buckling), $\phi_b M_n$ is the same as for compact section members as given in Equation 3.39 or Equation 3.40.

Noncompact Section Members Bent About Their Minor Axes
Regardless of the value of L_b, the limit state will be either flange or web local buckling, and $\phi_b M_n$ is given by Equation 3.42.

Slender Element Sections

Refer to the section on Plate Girder.

Tees and Double Angle Bent About Their Major Axes

The design flexural strength for tees and double-angle beams with flange and web slenderness ratios less than the corresponding limiting slenderness ratios λ_r shown in Table 3.8 is given by

$$\phi_b M_n = 0.90 \left[\frac{\pi \sqrt{EI_y GJ}}{L_b} (B + \sqrt{1 + B^2}) \right] \leq 0.90(CM_y) \tag{3.44}$$

where

$$B = \pm 2.3 \left(\frac{d}{L_b} \right) \sqrt{\frac{I_y}{J}} \tag{3.45}$$

$C = 1.5$ for stems in tension, and 1.0 for stems in compression.

Use the plus sign for B if the *entire* length of the stem along the unbraced length of the member is in tension. Otherwise, use the minus sign. The other variables in Equation 3.44 are defined as before in Equation 3.39.

Shear Strength Criterion

For a satisfactory design, the design shear strength of the webs must exceed the factored shear acting on the cross-section, i.e.,

$$\phi_v V_n \geq V_u \tag{3.46}$$

Depending on the slenderness ratios of the webs, three limit states can be identified: shear yielding, inelastic shear buckling, and elastic shear buckling. The design shear strength that corresponds to each of these limit states is given as follows:

For $h/t_w \leq 418/\sqrt{F_{yw}}$, (Shear yielding of web)

$$\phi_v V_n = 0.90[0.60 F_{yw} A_w] \tag{3.47}$$

For $418/\sqrt{F_{yw}} < h/t_w \leq 523/\sqrt{F_{yw}}$, (Inelastic shear buckling of web)

$$\phi_v V_n = 0.90 \left[0.60 F_{yw} A_w \frac{418/\sqrt{F_{yw}}}{h/t_w} \right] \tag{3.48}$$

For $523/\sqrt{F_{yw}} < h/t_w \leq 260$, (Elastic shear buckling of web)

$$\phi_v V_n = 0.90 \left[\frac{132{,}000 A_w}{(h/t_w)^2} \right] \tag{3.49}$$

The variables used in the above equations are defined in the following:

h = clear distance between flanges less the fillet or corner radius, in.
t_w = web thickness, in.
F_{yw} = yield stress of web, ksi
A_w = dt_w, in.^{2}
d = overall depth of section, in.

Criteria for Concentrated Loads

When concentrated loads are applied normal to the flanges in planes parallel to the webs of flexural members, the flange(s) and web(s) must be checked to ensure that they have sufficient strengths ϕR_n to withstand the concentrated forces R_u, i.e.,

$$\phi R_n \geq R_u \tag{3.50}$$

The design strength for a variety of limit states are given below:

Local Flange Bending
The design strength for local flange bending is given by

$$\phi R_n \geq 0.90[6.25t_f^2 F_{yf}] \tag{3.51}$$

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

Local Web Yielding
The design strength for yielding of a beam web at the toe of the fillet under tensile or compressive loads acting on one or both flanges are:

If the load acts at a distance from the beam end which exceeds the depth of the member

$$\phi R_n = 1.00[(5k + N)F_{yw}t_w] \tag{3.52}$$

If the load acts at a distance from the beam end which does not exceed the depth of the member

$$\phi R_n = 1.00[(2.5k + N)F_{yw}t_w] \tag{3.53}$$

where
k = distance from outer face of flange to web toe of fillet
N = length of bearing on the beam flange
F_{yw} = web yield stress
t_w = web thickness

Web Crippling
The design strength for crippling of a beam web under compressive loads acting on one or both flanges are:
If the load acts at a distance from the beam end which exceeds half the depth of the beam

$$\phi R_n = 0.75 \left\{ 135t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{F_{yw}t_f}{t_w}} \right\} \tag{3.54}$$

If the load acts at a distance from the beam end which does not exceed half the depth of the beam and if $N/d \leq 0.2$

$$\phi R_n = 0.75 \left\{ 68t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{F_{yw}t_f}{t_w}} \right\} \tag{3.55}$$

If the load acts at a distance from the beam end which does not exceed half the depth of the beam and if N/d>0.2

$$\phi R_n = 0.75 \left\{ 68t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw}t_f}{t_w}} \right\} \tag{3.56}$$

where

d = overall depth of the section, in.
t_f = flange thickness, in.

The other variables are the same as those defined in Equations 3.52 and 3.53.

Sidesway Web Buckling

Sidesway web buckling may occur in the web of a member if a compressive concentrated load is applied to a flange which is not restrained against relative movement by stiffeners or lateral bracings. The sidesway web buckling design strength for the member is:

If the loaded flange is restrained against rotation about the longitudinal member axis and $(h_c/t_w)(l/b_f) \leq 2.3$

$$\phi R_n = 0.85 \left\{ \frac{C_r t_w^3 t_f}{h^2} \left[1 + 0.4 \left(\frac{h/t_w}{l/b_f} \right)^3 \right] \right\} \tag{3.57}$$

If the loaded flange is not restrained against rotation about the longitudinal member axis and $(h_c/t_w)(l/b_f) \leq 1.7$

$$\phi R_n = 0.85 \left\{ \frac{C_r t_w^3 t_f}{h^2} \left[0.4 \left(\frac{h/t_w}{l/b_f} \right)^3 \right] \right\} \tag{3.58}$$

where

t_f = flange thickness, in.
t_w = web thickness, in.
h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or clear distance between flanges when welds are used for built-up shapes, in.
b_f = flange width, in.
l = largest laterally unbraced length along either flange at the point of load, in.
C_r = 960,000 if M_u/M_y <1 at the point of load, ksi
= 480,000 if M_u/M_y ≥1 at the point of load, ksi

Compression Buckling of the Web

This limit state may occur in members with unstiffened webs when both flanges are subjected to compressive forces. The design strength for this limit state is

$$\phi R_n = 0.90 \left[\frac{4,100t_w^3\sqrt{F_{yw}}}{h} \right] \tag{3.59}$$

This design strength shall be reduced by 50% if the concentrated forces are acting at a distance from the beam end which is half the beam depth. The variables in Equation 3.59 are the same as those defined in Equations 3.56 to 3.58.

Stiffeners shall be provided in pairs if any one of the above strength criteria is violated. If the local flange bending or the local web yielding criterion is violated, the stiffener pair to be provided to carry the excess R_u need not extend more than one-half the web depth. The stiffeners shall be welded to the loaded flange if the applied force is tensile. They shall either bear on or be welded to the loaded

flange if the applied force is compressive. If the web crippling or the compression web buckling criterion is violated, the stiffener pair to be provided shall extend the full height of the web. They shall be designed as axially loaded compression members (see section on Compression Members) with an effective length factor $K = 0.75$, a cross-section A_g composed of the cross-sectional areas of the stiffeners plus $25t_w^2$ for interior stiffeners, and $12t_w^2$ for stiffeners at member ends.

Deflection Criterion

The deflection criterion is the same as that for ASD. Since deflection is a serviceability limit state, service (rather than factored) loads should be used in deflection computations.

3.5.3 Continuous Beams

Continuous beams shall be designed in accordance with the criteria for flexural members given in the preceding section. However, a 10% reduction in negative moments due to gravity loads is allowed at the supports provided that:

1. the maximum positive moment between supports is increased by 1/10 the average of the negative moments at the supports;

2. the section is compact;

3. the lateral unbraced length does not exceed L_c (for ASD), or L_{pd} (for LRFD) where L_c is as defined in Equation 3.26 and L_{pd} is given by

$$L_{pd} = \begin{cases} \frac{3,600+2,200(M_1/M_2)}{F_y} r_y, & \text{for I-shaped members} \\ \frac{5,000+3,000(M_1/M_2)}{F_y} r_y, & \text{for solid rectangular and box sections} \end{cases} \tag{3.60}$$

in which

F_y = specified minimum yield stress of the compression flange, ksi

r_y = radius of gyration about the minor axis, in.

M_1/M_2 = ratio of smaller to larger moment within the unbraced length, taken as positive if the moments cause reverse curvature and negative if the moments cause single curvature.

4. the beam is not a hybrid member;

5. the beam is not made of high strength steel;

6. the beam is continuous over the supports (i.e., not cantilevered).

EXAMPLE 3.4:

Using LRFD, select the lightest W section for the three-span continuous beam shown in Figure 3.10a to support a uniformly distributed dead load of 1.5 k/ft (22 kN/m) and a uniformly distributed live load of 3 k/ft (44 kN/m). The beam is laterally braced at the supports A,B,C, and D. Use A36 steel.

Load combinations

The beam is to be designed based on the worst load combination of Table 3.3 By inspection, the load combination $1.2D+1.6L$ will control the design. Thus, the beam will be designed to support a factored uniformly distributed dead load of $1.2 \times 1.5 = 1.8$ k/ft and a factored uniformly distributed live load of $1.6 \times 3 = 4.8$ k/ft.

Placement of loads

The uniform dead load is to be applied over the entire length of the beam as shown in Figure 3.10b. The uniform live load is to be applied to spans AB and CD as shown in Figure 3.10c to obtain the
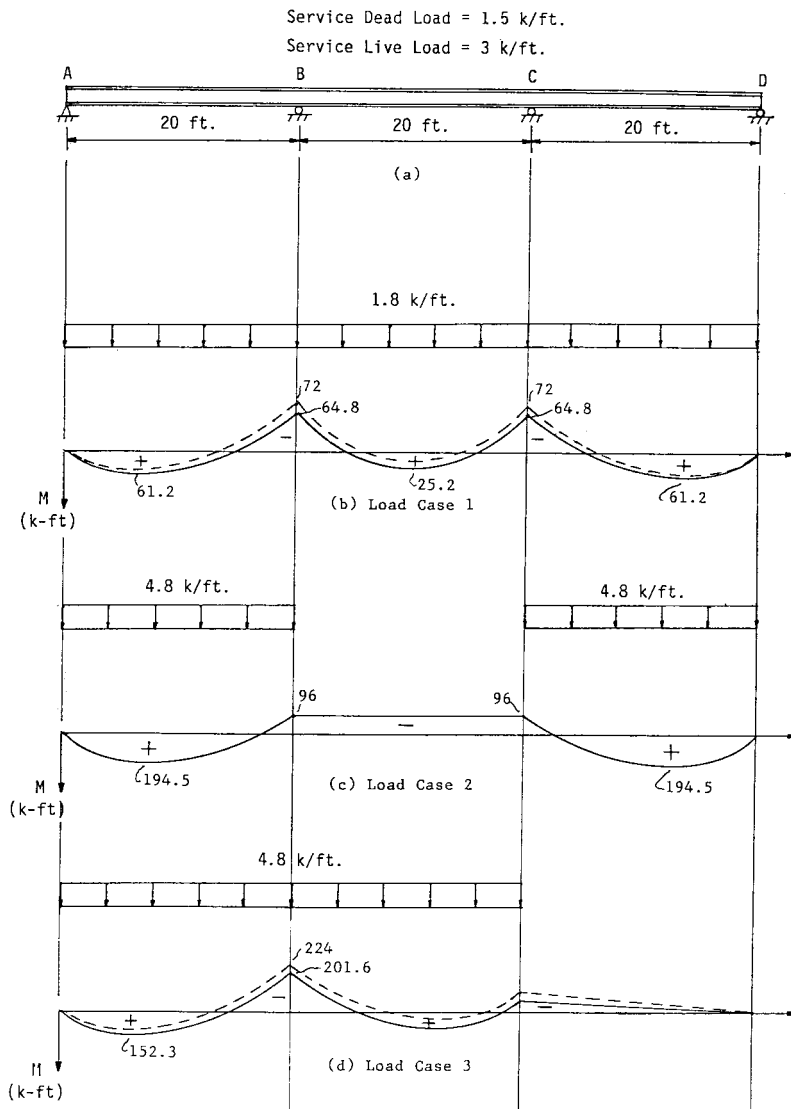
FIGURE 3.10: Design of a three-span continuous beam (1 k = 4.45 kN, 1 ft = 0.305 m).

maximum positive moment and it is to be applied to spans AB and BC as shown in Figure 3.10d to obtain the maximum negative moment.

Reduction of negative moment at supports

Assuming the beam is compact and $L_b < L_{pd}$ (we shall check these assumptions later), a 10% reduction in support moment due to gravity load is allowed provided that the maximum moment is increased by 1/10 the average of the negative support moments. This reduction is shown in the moment diagrams as solid lines in Figures 3.10b and 3.10d (The dotted lines in these figures represent the unadjusted moment diagrams). This provision for support moment reduction takes into consideration the beneficial effect of moment redistribution in continuous beams and it allows for the selection of a lighter section if the design is governed by negative moments. Note that no

reduction in negative moments is made to the case when only spans AB and CD are loaded. This is because for this load case, the negative support moments are less than the positive in-span moments.

Determination of the required flexural strength, M_u

Combining load case 1 and load case 2, the maximum positive moment is found to be 256 kip-ft. Combining load case 1 and load case 3, the maximum negative moment is found to be 266 kip-ft. Thus, the design will be controlled by the negative moment and so $M_u = 266$ kip-ft.

Beam selection

A beam section is to be selected based on Equation 3.36. The critical segment of the beam is span BC. For this span, the lateral unsupported length, L_b, is equal to 20 ft. For simplicity, the bending coefficient, C_b, is conservatively taken as 1. The selection of a beam section is facilitated by the use of a series of beam charts contained in the AISC-LRFD Manual [22]. Beam charts are plots of flexural design strength $\phi_b M_n$ of beams as a function of the lateral unsupported length L_b based on Equations 3.37 to 3.39. A beam is considered satisfactory for the limit state of flexure if the beam strength curve envelopes the required flexural strength for a given L_b.

For the present example, $L_b = 20$ ft. and $M_u = 266$ kip-ft, the lightest section (the first solid curve that envelopes $M_u = 266$ kip-ft for $L_b = 20$ ft) obtained from the chart is a W16x67 section. Upon adding the factored dead weight of this W16x67 section to the specified loads, the required strength increases from 266 kip-ft to 269 kip-ft. Nevertheless, the beam strength curve still envelopes this required strength for $L_b = 20$ ft; therefore, the section is adequate.

Check for compactness

For the W16x67 section,

$$
\begin{aligned}
\text{Flange:} \quad & \left[\frac{b_f}{2t_f} = 7.7\right] \quad < \quad \left[\frac{65}{\sqrt{F_y}} = 10.8\right] \\
\text{Web:} \quad & \left[\frac{h_c}{t_w} = 35.9\right] \quad < \quad \left[\frac{640}{\sqrt{F_y}} = 106.7\right]
\end{aligned}
$$

Therefore, the section is compact.

Check whether $L_b < L_{pd}$

Using Equation 3.64, with $M_1/M_2 = 0$, $r_y = 2.46$ in. and $F_y = 36$ ksi, we have $L_{pd} = 246$ in. (or 20.5 ft). Since $L_b = 20$ ft is less than $L_{pd} = 20.5$ ft, the assumption made earlier is validated.

Check for the limit state of shear

The selected section must satisfy the shear strength criterion of Equation 3.46. From structural analysis, it can be shown that maximum shear occurs just to the left of support B under load case 1 (for dead load) and load case 3 (for live load). It has a magnitude of 81.8 kips. For the W16x67 section, $h/t_w = 35.9$ which is less than $418/\sqrt{F_{yw}} = 69.7$, so the design shear strength is given by Equation 3.47. We have, for $F_{yw} = 36$ ksi and $A_w = dt_w = (16.33)(0.395)$,

$$[\phi_v V_n = 0.90(0.60 F_{yw} A_w) = 125 \text{ kips }] > [V_u = 81.8 \text{ kips }]$$

Therefore, shear is not a concern. Normally, the limit state of shear will not be controlled unless for short beams subjected to heavy loads.

Check for limit state of deflection

Deflection is a serviceability limit state. As a result, a designer should use service (not factored) loads, for deflection calculations. In addition, most beams are cambered to offset deflection caused by dead loads, so only live loads are considered in deflection calculations. From structural analysis, it can be shown that maximum deflection occurs in span AB and CD when (service) live loads are placed on those two spans. The magnitude of the deflection is 0.297 in. Assuming the maximum

allowable deflection is $L/360$ where L is the span length between supports, we have an allowable deflection of $20 \times 12/360 = 0.667$ in. Since the calculated deflection is less than the allowable deflection, deflection is not a problem.

Check for the limit state of web yielding and web crippling at points of concentrated loads

From a structural analysis it can be shown that maximum support reaction occurs at support B when the beam is subjected to loads shown as load case 1 (for dead load) and load case 3 (for live load). The magnitude of the reaction R_u is 157 kips. Assuming point bearing, i.e., $N = 0$, we have, for $d = 16.33$ in., $k = 1.375$ in., $t_f = 0.665$ in., and $t_w = 0.395$ in.,

Web Yielding: $\left[\phi R_n = \text{ Equation } 3.52 \quad = 97.8 \quad \text{kips} \right] \quad < \quad \left[R_u = 157 \quad \text{kips} \right]$
Web Crippling: $\left[\phi R_n = \text{ Equation } 3.54 \quad = 123 \quad \text{kips} \right] \quad < \quad \left[R_u = 157 \quad \text{kips} \right]$

Thus, both the web yielding and web crippling criteria are violated. As a result, we need to provide web stiffeners or bearing plate at support B. Suppose we choose the latter, the size of the bearing plate is to be determined by solving Equation 3.52 and Equation 3.54 for N, given $R_u = 157$ kips. Solving Equation 3.52 and Equation 3.54 for N, we obtain $N = 4.2$ in. and 3.3 in., respectively. So, use $N = 4.25$ in. The width of the plate, B, should conform with the flange width, b_f, of the W-section. The W16x67 section has a flange width of 10.235 in., so use $B = 10.5$ in. For uniformity, use the same size plate at all the supports. The bearing plates are to be welded to the supporting flange of the W-section.

Use a W16x67 section. Provide bearing plates of size $10.5'' \times 4''$ at the supports.

3.5.4 Lateral Bracing of Beams

The design strength of beams that bent about their major axes depends on their lateral unsupported length, L_b. The manner a beam is braced against out-of-plane deformation affects its design. Bracing can be provided by various means such as cross beams, or diaphragms, or encasement of the compression flange of the beam in the floor slab. Although neither the ASD nor the LRFD specification addresses the design of braces, a number of methodologies have been proposed in the literature for the design of braces [30]. It is important to note that braces must be designed with sufficient strength and stiffness to prevent out-of-plane movement of the beam at the braced points. In what follows, the requirements for brace design as specified in the Canadian Standards Association [10] for the limit states design of steel structures will be given.

Stiffness Requirement:

The stiffness of the bracing assembly in a direction perpendicular to the longitudinal axis of the braced member, in the plane of buckling, must exceed

$$k_b = \frac{\beta C_f}{L_b}[1 + \frac{\Delta_o}{\Delta_b}] \tag{3.61}$$

where

β = bracing coefficient given by 2, 3, 3.41, and 3.63 for one, two, three, and four or more equally spaced braces, respectively.

C_f = compressive force in the braced member. In a limit state design, C_f for a doubly symmetric I-shaped beam can be calculated by dividing its design flexural strength, $\phi_b M_p$, by the distance between centroids of the flanges, $d - t_f$.

L_b = lateral unsupported length of the braced member.

Δ_o = initial misalignment of the braced member at the point of the brace. Δ_o may be taken as the sweep tolerance of the braced member.

Δ_b = displacement of the braced member and bracing assembly under force C_f at the point of the brace. For a trial design, Δ_b may be taken as Δ_o.

Strength Requirement:

In addition to the stiffness requirement as stipulated above, braces must be designed for strength that exceed

$$P_b = k_b \Delta_b \qquad (3.62)$$

where P_b is the force in the bracing assembly under factored loads.

If a series of parallel members are being braced, a reduced initial misalignment can be used in Equation 3.61 to account for probable force redistribution when the bracing assembly transfers force from one braced member to another. This reduced misalignment is given by

$$\Delta_m = \left[0.2 + \frac{0.8}{\sqrt{n}} \right] \Delta_o \qquad (3.63)$$

where n is the number of parallel braced members.

Finally, if the brace is under compression, a maximum slenderness, l/r, not exceeding 200 is recommended.

EXAMPLE 3.5:

Design an I-shaped cross beam 15 ft (4.6 m) in length to be used as lateral braces to brace a 30-ft (9.1 m) long W30x90 beam at every third point. The sweep of the W30x90 section is 0.36 in. (9 mm) in the plane of the brace. A36 steel is used.

If a brace is provided at every third point, L_b for the W30x90 section is 10 ft. Therefore, the design flexural strength, $\phi_b M_n$, is 8890 kip-in, from which $C_f = \phi_b M_n/(d-t_f) = 8890/(29.53-0.610) = 307$ kips. As a first trial, assume $\Delta_b = \Delta_o = 0.36$ in., we can calculate from Equation 3.61 and Equation 3.62 the minimum stiffness and strength requirements for the cross beam

$$
\begin{aligned}
k_b &= \frac{3(307)}{10 \times 12}[1 + \frac{0.36}{0.36}] = 15.4 \text{ kips/in.} \\
P_b &= (15.4)(0.36) = 5.5 \text{ kips}
\end{aligned}
$$

Since the cross beam will be subject to compression, its slenderness ratio, l/r, should not exceed 200.

Try a W4x13 section ($A = 3.83$ in^2, $r_y = 1.00$ in., $\phi_c P_n = 25$ kips)

Stiffness,	$\frac{EA}{l}$	$= \frac{(29000)(3.83)}{15 \times 12} = 617$ kips/in.	> 15.4 kips/in.
Strength,	$\phi_c P_n$	$= 25$ kips	> 5.5 kips
Slenderness,	$\frac{l}{r_y}$	$= \frac{15 \times 12}{1.00} = 180$	< 200

Recalculate Δ_b using $P_b = 5.5$ kips and check adequacy of the W4x13 section.

$$
\begin{aligned}
\Delta_b &= \frac{P_b l}{EA} = \frac{(5.5)(15 \times 12)}{(29000)(3.83)} = 0.0089 \text{ in.} \\
k_b &= \frac{3(307)}{10 \times 12}\left[1 + \frac{0.36}{0.0089} \right] = 318 \text{ kips/in.} \quad < 617 \text{ kips/in. OK} \\
P_b &= (318)(0.0089) = 2.83 \text{ kips} \quad < 25 \text{ kips OK}
\end{aligned}
$$

Recalculate Δ_b using $P_b = 2.83$ kips and check adequacy of the W4x13 section.

$$\Delta_b = \frac{P_b l}{EA} = \frac{(2.83)(15 \times 12)}{(29000)(3.83)} = 0.0046 \text{ in.}$$

$$k_b = \frac{3(307)}{10 \times 12}\left[1 + \frac{0.36}{0.0046}\right] = 608 \;\; \text{kips/in.} \quad < 617 \;\; \text{kips/in.} \quad \text{OK}$$

$$P_b = (608)(0.0046) = 2.80 \;\; \text{kips} \quad < 25 \;\; \text{kips} \quad \text{OK}$$

Since P_b, and hence Δ_b, has converged, no more iterations are necessary.

Use W4x13 cross beams as braces.

3.6 Combined Flexure and Axial Force

When a member is subject to the combined action of bending and axial force, it must be designed to resist stresses and forces arising from both bending and axial actions. While a tensile axial force may induce a stiffening effect on the member, a compressive axial force tends to destabilize the member, and the instability effects due to member instability (P-δ effect) and frame instability (P-Δ effect) must be properly accounted for. P-δ effect arises when the axial force acts through the lateral deflection of the member relative to its chord. P-Δ effect arises when the axial force acts through the relative displacements of the two ends of the member. Both effects tend to increase member deflection and moment, and so they must be considered in the design. A number of approaches are available in the literature to handle these so-called P-delta effects (see for example [9, 13]). The design of members subject to combined bending and axial force is facilitated by the use of interaction equations. In these equations, the effects of bending and axial actions are combined in a certain manner to reflect the capacity demand on the member.

3.6.1 Allowable Stress Design

The interaction equations are:

If the axial force is tensile:

$$\frac{f_a}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \tag{3.64}$$

where

f_a	=	computed axial tensile stress
f_{bx}, f_{by}	=	computed bending tensile stresses about the major and minor axes, respectively
F_{bx}, F_{by}	=	allowable bending stresses about the major and minor axes, respectively (see section on Flexural Members)
F_t	=	allowable tensile stress (see section on Tension Members)

If the axial force is compressive:
Stability requirement

$$\frac{f_a}{F_a} + \left[\frac{C_{mx}}{\left(1 - \frac{f_a}{F'_{ex}}\right)}\right]\frac{f_{bx}}{F_{bx}} + \left[\frac{C_{my}}{\left(1 - \frac{f_a}{F'_{ey}}\right)}\right]\frac{f_{by}}{F_{by}} \leq 1.0 \tag{3.65}$$

Yield requirement

$$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \tag{3.66}$$

However, if the axial force is small (when $f_a/F_a \leq 0.15$), the following interaction equation can be used in lieu of the above equations.

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \tag{3.67}$$

The terms in the Equations 3.65 to 3.67 are defined as follows:

f_a, f_{bx}, f_{by} = computed axial compressive stress, computed bending stresses about the major and minor axes, respectively. These stresses are to be computed based on a *first-order analysis*

F_y = minimum specified yield stress

F'_{ex}, F'_{ey} = Euler stresses about the major and minor axes $(\pi^2 E/(Kl/r)_x, \pi^2 E/(Kl/r)_y)$ divided by a factor of safety of 23/12

C_m = a coefficient to account for the effect of moment gradient on member and frame instabilities (C_m is defined in the section on LRFD to follow)

The other terms are defined as in Equation 3.64.

The terms in brackets in Equation 3.65 are moment magnification factors. The computed bending stresses f_{bx}, f_{by} are magnified by these magnification factors to account for the P-delta effects in the member.

3.6.2 Load and Resistance Factor Design

Doubly or singly symmetric members subject to combined flexure and axial force shall be designed in accordance with the following interaction equations:

For $P_u/\phi P_n \geq 0.2$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \tag{3.68}$$

For $P_u/\phi P_n < 0.2$

$$\frac{P_u}{2\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \tag{3.69}$$

where, if P is tensile

P_u = factored tensile axial force

P_n = design tensile strength (see section on Tension Members)

M_u = factored moment (preferably obtained from a second-order analysis)

M_n = design flexural strength (see section on Flexural Members)

$\phi = \phi_t$ = resistance factor for tension = 0.90

ϕ_b = resistance factor for flexure = 0.90

and, if P is compressive

P_u = factored compressive axial force

P_n = design compressive strength (see section on Compression Members)

M_u = required flexural strength (see discussion below)

M_n = design flexural strength (see section on Flexural Members)

$\phi = \phi_c$ = resistance factor for compression = 0.85

ϕ_b = resistance factor for flexure = 0.90

The required flexural strength M_u shall be determined from a second-order elastic analysis. In lieu of such an analysis, the following equation may be used

$$M_u = B_1 M_{nt} + B_2 M_{lt} \tag{3.70}$$

where

M_{nt} = factored moment in member assuming the frame does not undergo lateral translation (see Figure 3.11)

M_{lt} = factored moment in a member as a result of lateral translation (see Figure 3.11)

B_1 = $C_m/(1 - P_u/P_e) \geq 1.0$ is the P-δ moment magnification factor

$$
\begin{aligned}
P_e &= \pi^2 E I/(KL)^2, \text{ with } K \le 1.0 \text{ in the plane of bending}\\
C_m &= \text{a coefficient to account for moment gradient, determined from the following discussion}\\
B_2 &= 1/[1 - (\Sigma P_u \Delta_{oh}/\Sigma H L)] \text{ or } B_2 = 1/[1 - (\Sigma P_u/\Sigma P_e)]\\
\Sigma P_u &= \text{sum of all factored loads acting on and above the story under consideration}\\
\Delta_{oh} &= \text{first-order interstory translation}\\
\Sigma H &= \text{sum of all lateral loads acting on and above the story under consideration}\\
L &= \text{story height}\\
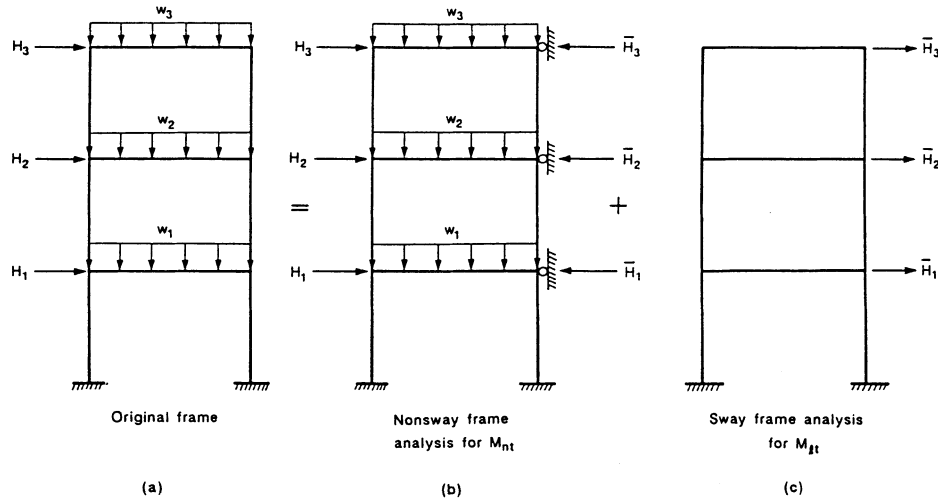P_e &= \pi^2 E I/(KL)^2
\end{aligned}
$$

FIGURE 3.11: Calculation of M_{nt} and M_{lt}.

For end-restrained members which do not undergo relative joint translation and are not subject to transverse loading between their supports in the plane of bending, C_m is given by

$$
C_m = 0.6 - 0.4 \left(\frac{M_1}{M_2} \right)
$$

where M_1/M_2 is the ratio of the smaller to larger member end moments. The ratio is positive if the member bends in reverse curvature and negative if the member bends in single curvature.

For end-restrained members which do not undergo relative joint translation and are subject to transverse loading between their supports in the plane of bending

$$
C_m = 0.85
$$

For unrestrained members which do not undergo relative joint translation and are subject to transverse loading between their supports in the plane of bending

$$
C_m = 1.00
$$

The selection of trial sections for use as beam-columns is facilitated by rewriting the interaction equations of Equation 3.68 and 3.69 into the so-called equivalent axial load form:

For $P_u/\phi_c P_n > 0.2$

$$
P_u + m_x M_{ux} + m_y U M_{uy} \le \phi_c P_n \tag{3.71}
$$

For $P_u/\phi_c P_n \leq 0.2$

$$\frac{P_u}{2} + \frac{9}{8}m_x M_{ux} + \frac{9}{8}m_y U M_{uy} \leq \phi_c P_n \tag{3.72}$$

where
$$m_x \quad = \quad (8/9)(\phi_c P_n/\phi_b M_{nx})$$
$$m_y U \quad = \quad (8/9)(\phi_c P_n/\phi_b M_{ny})$$

Numerical values for m and U are provided in the AISC Manual [22]. The advantage of using Equations 3.71 and 3.72 for preliminary design is that the terms on the left-hand side of the inequality can be regarded as an equivalent axial load, $(P_u)_{eff}$, thus allowing the designer to take advantage of the column tables provided in the manual for selecting trial sections.

3.7   Biaxial Bending

Members subjected to bending about both principal axes (e.g., purlins on an inclined roof) should be designed for biaxial bending. Since both moment about the major axis M_{ux} and moment about the minor axis M_{uy} create flexural stresses over the cross-section of the member, the design must take into consideration this stress combination.

3.7.1   Allowable Stress Design

The following interaction equation is often used for the design of beams subject to biaxial bending

$$f_{bx} + f_{by} \leq 0.60 F_y$$
or,
$$\frac{M_x}{S_x} + \frac{M_y}{S_y} \leq 0.60 F_y \tag{3.73}$$

where
$$M_x, M_y \quad = \quad \text{service load moments about the major and minor axes, respectively}$$
$$S_x, S_y \quad = \quad \text{elastic section moduli about the major and minor axes, respectively}$$
$$F_y \quad = \quad \text{specified minimum yield stress}$$

EXAMPLE 3.6:

Using ASD, select a W section to carry dead load moments $M_x = 20$ k-ft (27 kN-m) and $M_y = 5$ k-ft (6.8 kN-m), and live load moments $M_x = 50$ k-ft (68 kN-m) and $M_y = 15$ k-ft (20 kN-m). Use steel having $F_y = 50$ ksi (345 MPa).

Calculate service load moments:

$$M_x = M_{x,\text{dead}} + M_{x,\text{live}} = 20 + 50 = 70 \quad \text{k-ft}$$
$$M_y = M_{y,\text{dead}} = M_{y,\text{live}} = 5 + 15 = 20 \quad \text{k-ft}$$

Select section:

Substituting the above service load moments into Equation 3.73, we have

$$\frac{70 \times 12}{S_x} + \frac{20 \times 12}{S_y} \leq 0.60(50) \quad \text{or,} \quad 840 + 240\frac{S_x}{S_y} \leq 30 S_x$$

For W sections with depth below 14 in. the value of S_x/S_y normally falls in the range 3 to 8, and for W sections with depth above 14 in. the value of S_x/S_y normally falls in the range 5 to 12. Assuming

$S_x/S_y = 10$, we have from the above equation, $S_x \geq 108$ in.^{3}. Using the Allowable Stress Design Selection Table in the AISC-ASD Manual, lets try a W24x55 section ($S_x = 114$ in.^{3}, $S_y = 8.30$ in.^{3}). For the W24x55 section

$$\left[840 + 240\frac{114}{8.30} = 4136\right] > [30S_x = 30(114) = 3420] \quad \therefore \quad \text{NG}$$

The next lightest section is W21x62 ($S_x = 127$ in.^{3}, $S_y = 13.9$ in.^{3}). For this section

$$\left[840 + 240\frac{127}{13.9} = 3033\right] < [30S_x = 30(127) = 3810] \quad \therefore \quad \text{OK}$$

Use a W21x62 section.

3.7.2 Load and Resistance Factor Design

To avoid distress at the most severely stressed point, the following equation for the yielding limit state must be satisfied:
$$f_{un} \leq \phi_b F_y \tag{3.74}$$

where

f_{un}	$=$	$M_{ux}/S_x + M_{uy}/S_y$ is the flexural stress under factored loads
S_x, S_y	$=$	are the elastic section moduli about the major and minor axes, respectively
ϕ_b	$=$	0.90
F_y	$=$	specified minimum yield stress

In addition, the limit state for lateral torsional buckling about the major axis should also be checked, i.e.,
$$\phi_b M_{nx} \geq M_{ux} \tag{3.75}$$

$\phi_b M_{nx}$ is the design flexural strength about the major axis (see section on Flexural Members). Note that lateral torsional buckling will not occur about the minor axis. Equation 3.74 can be rearranged to give:
$$S_x \leq \frac{M_{ux}}{\phi_b F_y} + \frac{M_{uy}}{\phi_b F_y}\left(\frac{S_x}{S_y}\right) \approx \frac{M_{ux}}{\phi_b F_y} + \frac{M_{uy}}{\phi_b F_y}\left(3.5\frac{d}{b_f}\right) \tag{3.76}$$

The approximation $(S_x/S_y) \approx (3.5d/b_f)$ where d is the overall depth and b_f is the flange width was suggested by Gaylord et al. [15] for doubly symmetric I-shaped sections. The use of Equation 3.74 greatly facilitates the selection of trial sections for use in biaxial bending problems.

3.8 Combined Bending, Torsion, and Axial Force

Members subjected to the combined effect of bending, torsion, and axial force should be designed to satisfy the following limit states:

Yielding under normal stress
$$\phi F_y \geq f_{un} \tag{3.77}$$

where

ϕ	$=$	0.90
F_y	$=$	specified minimum yield stress
f_{un}	$=$	maximum normal stress determined from an elastic analysis under factored loads

Yielding under shear stress
$$\phi(0.6F_y) \geq f_{uv} \tag{3.78}$$

where

ϕ $=$ 0.90

F_y $=$ specified minimum yield stress

f_{uv} $=$ maximum shear stress determined from an elastic analysis under factored loads

Buckling

$$\phi_c F_{cr} \geq f_{un} \quad \text{or} \quad f_{uv}, \quad \text{whichever is applicable} \tag{3.79}$$

where

$\phi_c F_{cr}$ $=$ $\phi_c P_n / A_g$, in which $\phi_c P_n$ is the design compressive strength of the member (see section on Compression Members) and A_g is the gross cross-section area

f_{un}, f_{uv} $=$ normal and shear stresses as defined in Equation 3.77 and 3.78

3.9 Frames

Frames are designed as a collection of structural components such as beams, beam-columns (columns), and connections. According to the restraint characteristics of the connections used in the construction, frames can be designed as Type I (rigid framing), Type II (simple framing), Type III (semi-rigid framing) in ASD, or fully restrained (rigid), partially restrained (semi-rigid) in LRFD. The design of rigid frames necessitates the use of connections capable of transmitting the full or a significant portion of the moment developed between the connecting members. The rigidity of the connections must be such that the angles between intersecting members should remain virtually unchanged under factored loads. The design of semi-rigid frames is permitted upon evidence of the connections to deliver a predicable amount of moment restraint. The main members joined by these connections must be designed to assure that their ultimate capacities will not exceed those of the connections. The design of simple frames is based on the assumption that the connections provide no moment restraint to the beam insofar as gravity loads are concerned but these connections should have adequate capacity to resist wind moments. Semi-rigid and simple framings often incur inelastic deformation in the connections. The connections used in these constructions must be proportioned to possess sufficient ductility to avoid overstress of the fasteners or welds.

Regardless of the types of constructions used, due consideration must be given to account for member and frame instability (P-δ and P-Δ) effects either by the use of a second-order analysis or by other means such as moment magnification factors. The end-restrained effect on members should also be accounted for by the use of the effective length factor (see Chapter 17).

Frames can be designed as sidesway inhibited (braced) or sidesway uninhibited (unbraced). In sidesway inhibited frames, frame drift is controlled by the presence of a bracing system (e.g., shear walls, diagonal or cross braces, etc.). In sidesway uninhibited frames, frame drift is limited by the flexural rigidity of the connected members and diaphragm action of the floors. Most sidesway uninhibited frames are designed as Type I or Type FR frames using moment connections. Under normal circumstances, the amount of interstory drift under service loads should not exceed $h/500$ to $h/300$ where h is the story height. Higher value of interstory drift is allowed only if it does not create serviceability concerns.

Beams in sidesway inhibited frames are often subject to high axial forces. As a result, they should be designed as beam-columns using beam-column interaction equations. Furthermore, vertical bracing systems should be provided for braced multistory frames to prevent vertical buckling of the frames under gravity loads.

3.10 Plate Girders

Plate girders are built-up beams. They are used as flexural members to carry extremely large lateral loads. A flexural member is considered as a plate girder if the width-thickness ratio of the web, h_c/t_w, exceeds $760/\sqrt{F_b}$ (F_b = allowable flexural stress) according to ASD, or $970/\sqrt{F_{yf}}$ (F_{yf} = minimum specified flange yield stress) according to LRFD. Because of the large web slenderness, plate girders are often designed with transverse stiffeners to reinforce the web and to allow for post-buckling (shear) strength (i.e., tension field action) to develop. Table 3.9 summarizes the requirements for transverse stiffeners for plate girders based on the web slenderness ratio h/t_w. Two types of transverse stiffeners are used for plate girders: bearing stiffeners and intermediate stiffeners. Bearing stiffeners are used at unframed girder ends and at concentrated load points where the web yielding or web crippling criterion is violated. Bearing stiffeners extend the full depth of the web from the bottom of the top flange to the top of the bottom flange. Intermediate stiffeners are used when the width-thickness ratio of the web, h/t_w, exceeds 260, or when the shear criterion is violated, or when tension field action is considered in the design. Intermediate stiffeners need not extend the full depth of the web but must be in contact with the compression flange of the girder.

Normally, the depths of plate girder sections are so large that simple beam theory which postulates that plane sections before bending remain plane after bending does not apply. As a result, a different set of design formulas for plate girders are required.

TABLE 3.9 Web Stiffeners Requirements

Range of web slenderness	Stiffeners requirements
$\dfrac{h}{t_w} \leq 260$	Plate girder can be designed without web stiffeners.
$260 \leq \dfrac{h}{t_w} \leq \dfrac{14{,}000}{\sqrt{F_{yf}(F_{yf}+16.5)}}$	Plate girder must be designed with web stiffeners. The spacing of stiffeners, a, can exceed $1.5h$. The actual spacing is determined by the shear criterion.
$\dfrac{14{,}000}{\sqrt{F_{yf}(F_{yf}+16.5)}} < \dfrac{h}{t_w} \leq \dfrac{2{,}000}{\sqrt{F_{yf}}}$	Plate girder must be designed with web stiffeners. The spacing of stiffeners, a, cannot exceed $1.5h$.

a	=	clear distance between stiffeners
h	=	clear distance between flanges when welds are used or the distance between adjacent lines of fasteners when bolts are used
t_w	=	web thickness
F_{yf}	=	compression flange yield stress, ksi

3.10.1 Allowable Stress Design

Allowable Bending Stress

The maximum bending stress in the compression flange of the girder computed using the flexure formula shall not exceed the allowable value, F'_b, given by

$$F'_b = F_b R_{PG} R_e \tag{3.80}$$

where

F_b	=	applicable allowable bending stress as discussed in the section on Flexural Members
R_{PG}	=	plate girder stress reduction factor $= 1 - 0.0005(A_w/A_f)(h/t_w - 760/\sqrt{F_b}) \leq 1.0$
R_e	=	hybrid girder factor $= [12 + (A_w/A_f)(3\alpha - \alpha^3)]/[12 + 2(A_w/A_f)] \leq 1.0$, $R_e = 1$ for non-hybrid girder
A_w	=	area of web
A_f	=	area of compression flange

$$\alpha \quad = \quad 0.60F_{yw}/F_b \leq 1.0$$
$$F_{yw} \quad = \quad \text{yield stress of web}$$

Allowable Shear Stress

Without tension field action:
The allowable shear stress is the same as that for beams given in Equation 3.33.

With tension field action:
The allowable shear stress is given by

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right] \leq 0.40F_y \tag{3.81}$$

Note that tension field action can be considered in the design only for non-hybrid girders. If tension field action is considered, transverse stiffeners must be provided and spaced at a distance such that the computed average web shear stress, f_v, obtained by dividing the total shear by the web area does not exceed the allowable shear stress, F_v, given by Equation 3.81. In addition, the computed bending tensile stress in the panel where tension field action is considered cannot exceed $0.60F_y$, nor $(0.825 - 0.375\,f_v/F_v)F_y$ where f_v is the computed average web shear stress and F_v is the allowable web shear stress given in Equation 3.81. The shear transfer criterion given by Equation 3.84 must also be satisfied.

Transverse Stiffeners

Transverse stiffeners must be designed to satisfy the following criteria.

Moment of inertial criterion:
With reference to an axis in the plane of the web, the moment of inertia of the stiffeners, in cubic inches, shall satisfy the condition

$$I_{st} \geq \left(\frac{h}{50}\right)^4 \tag{3.82}$$

where h is the clear distance between flanges, in inches.

Area criterion:
The total area of the stiffeners, in square inches, shall satisfy the condition

$$A_{st} \geq \frac{1 - C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{\sqrt{1 + (a/h)^2}}\right]YDht_w \tag{3.83}$$

where
$C_v \quad = \quad$ shear buckling coefficient as defined in Equation 3.33
$a \quad = \quad$ stiffeners' spacing
$h \quad = \quad$ clear distance between flanges
$t_w \quad = \quad$ web thickness, in.
$Y \quad = \quad$ ratio of web yield stress to stiffener yield stress
$D \quad = \quad$ 1.0 for stiffeners furnished in pairs, 1.8 for single angle stiffeners, and 2.4 for single plate stiffeners

Shear transfer criterion:
If tension field action is considered, the total shear transfer, in kips/in., of the stiffeners shall not be less than

$$f_{vs} = h\sqrt{\left(\frac{F_{yw}}{340}\right)^3} \tag{3.84}$$

where

F_{yw} = web yield stress, ksi

h = clear distance between flanges, in.

The value of f_{vs} can be reduced proportionally if the computed average web shear stress, f_v, is less than F_v given in Equation 3.81.

3.10.2 Load and Resistance Factor Design

Flexural Strength Criterion

Doubly or singly symmetric single-web plate girders loaded in the plane of the web should satisfy the flexural strength criterion of Equation 3.36. The plate girder design flexural strength is given by:

For the limit state of tension flange yielding

$$\phi_b M_n = 0.90[S_{xt} R_e F_{yt}] \tag{3.85}$$

For the limit state of compression flange buckling

$$\phi_b M_n = 0.90[S_{xc} R_{PG} R_e F_{cr}] \tag{3.86}$$

where

S_{xt} = section modulus referred to the tension flange = I_x/c_t

S_{xc} = section modulus referred to the compression flange = I_x/c_c

I_x = moment of inertia about the major axis

c_t = distance from neutral axis to extreme fiber of the tension flange

c_c = distance from neutral axis to extreme fiber of the compression flange

R_{PG} = plate girder bending strength reduction factor = $1 - a_r(h_c/t_w - 970/\sqrt{F_{cr}})/(1,200 + 300a_r) \leq 1.0$

R_e = hybrid girder factor = $[12 + a_r(3m - m^3)]/(12 + 2a_r) \leq 1.0$ ($R_e = 1$ for non-hybrid girder)

a_r = ratio of web area to compression flange area

m = ratio of web yield stress to flange yield stress or F_{cr}

F_{yt} = tension flange yield stress

F_{cr} = critical compression flange stress calculated as follows:

Limit state	Range of slenderness	F_{cr}
Flange local buckling	$\dfrac{b_f}{2t_f} \le \dfrac{65}{\sqrt{F_{yf}}}$	F_{yf}
	$\dfrac{65}{\sqrt{F_{yf}}} < \dfrac{b_f}{2t_f} \le \dfrac{230}{\sqrt{F_{yf}/k_c}}$	$F_{yf}\left[1 - \dfrac{1}{2}\left(\dfrac{\frac{b_f}{2t_f} - \frac{65}{\sqrt{F_{yf}}}}{\frac{230}{\sqrt{F_{yf}/k_c}} - \frac{64}{\sqrt{F_{yf}}}}\right)\right] \le F_{yf}$
	$\dfrac{b_f}{2t_f} > \dfrac{230}{\sqrt{F_{yf}/k_c}}$	$\dfrac{26{,}200k_c}{\left(\frac{b_f}{2t_f}\right)^2}$
Lateral torsional buckling	$\dfrac{L_b}{r_T} \le \dfrac{300}{\sqrt{F_{yf}}}$	F_{yf}
	$\dfrac{300}{\sqrt{F_{yf}}} < \dfrac{L_b}{r_T} \le \dfrac{756}{\sqrt{F_{yf}}}$	$C_b F_{yf}\left[1 - \dfrac{1}{2}\left(\dfrac{\frac{L_b}{r_T} - \frac{300}{\sqrt{F_{yf}}}}{\frac{756}{\sqrt{F_{yf}}} - \frac{300}{\sqrt{F_{yf}}}}\right)\right] \le F_{yf}$
	$\dfrac{L_b}{r_T} > \dfrac{756}{\sqrt{F_{yf}}}$	$\dfrac{286{,}000C_b}{\left(\frac{L_b}{r_T}\right)^2}$

k_c = $4/\sqrt{(h/t_w)}, 0.35 \le k_c \le 0.763$

b_f = compression flange width

t_f = compression flange thickness

L_b = lateral unbraced length of the girder

r_T = $\sqrt{[(t_f b_f^3/12 + h_c t_w^3/72)/(b_f t_f + h_c t_w/6)]}$

h_c = twice the distance from the neutral axis to the inside face of the compression flange less the fillet

t_w = web thickness

F_{yf} = yield stress of compression flange, ksi

C_b = Bending coefficient (see section on Flexural Members)

F_{cr} must be calculated for both flange local buckling and lateral torsional buckling. The smaller value of F_{cr} is used in Equation 3.86.

The plate girder bending strength reduction factor R_{PG} is a factor to account for the nonlinear flexural stress distribution along the depth of the girder. The hybrid girder factor is a reduction factor to account for the lower yield strength of the web when the nominal moment capacity is computed assuming a homogeneous section made entirely of the higher yield stress of the flange.

Shear Strength Criterion

Plate girders can be designed with or without the consideration of tension field action. If tension field action is considered, intermediate web stiffeners must be provided and spaced at a distance, a, such that a/h is smaller than 3 or $[260/(h/t_w)]^2$, whichever is smaller. Also, one must check the flexure-shear interaction of Equation 3.89, if appropriate. Consideration of tension field action is not allowed if (1) the panel is an end panel, (2) the plate girder is a hybrid girder, (3) the plate girder is a web tapered girder, or (4) a/h exceeds 3 or $[260/(h/t_w)]^2$, whichever is smaller.

The design shear strength, $\phi_v V_n$, of a plate girder is determined as follows:

If tension field action is not considered:
$\phi_v V_n$ are the same as those for beams as given in Equations 3.47 to 3.49.

If tension field action is considered and $h/t_w \le 187/\sqrt{(k_v/F_{yw})}$:

$$\phi_v V_n = 0.90[0.60A_w F_{yw}] \tag{3.87}$$

and, if $h/t_w > 187/\sqrt{(k_v/F_{yw})}$:

$$\phi_v V_n = 0.90 \left[0.60 A_w F_{yw} \left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \right] \qquad (3.88)$$

where

k_v = $5 + 5/(a/h)^2$ (k_v shall be taken as 5.0 if a/h exceeds 3.0 or $[260/(h/t_w)]^2$, whichever is smaller)

A_w = dt_w

F_{yw} =   web yield stress, ksi

C_v =   shear coefficient, calculated as follows:

Range of h/t_w	C_v
$187\sqrt{\frac{k_v}{F_{yw}}} \le \frac{h}{t_w} \le 234\sqrt{\frac{k_v}{F_{yw}}}$	$\frac{187\sqrt{k_v/F_{yw}}}{h/t_w}$
$\frac{h}{t_w} > 234\sqrt{\frac{k_v}{F_{yw}}}$	$\frac{44{,}000 k_v}{(h/t_w)^2 F_{yw}}$

Flexure-Shear Interaction

Plate girders designed for tension field action must satisfy the flexure-shear interaction criterion in regions where $0.60\phi V_n \le V_u \le \phi V_n$ and $0.75\phi M_n \le M_u \le \phi M_n$

$$\frac{M_u}{\phi M_n} + 0.625 \frac{V_u}{\phi V_n} \le 1.375 \qquad (3.89)$$

where $\phi = 0.90$.

Bearing Stiffeners

Bearing stiffeners must be provided for a plate girder at unframed girder ends and at points of concentrated loads where the web yielding or the web crippling criterion is violated (see section on Concentrated Load Criteria). Bearing stiffeners shall be provided in pairs and extended from the upper flange to the lower flange of the girder. Denoting b_{st} as the width of one stiffener and t_{st} as its thickness, bearing stiffeners shall be portioned to satisfy the following limit states:

For the limit state of local buckling

$$\frac{b_{st}}{t_{st}} \le \frac{95}{\sqrt{F_y}} \qquad (3.90)$$

For the limit state of compression

The design compressive strength, $\phi_c P_n$, must exceed the required compressive force acting on the stiffeners. $\phi_c P_n$ is to be determined based on an effective length factor K of 0.75 and an effective area, A_{eff}, equal to the area of the bearing stiffeners plus a portion of the web. For end bearing, this effective area is equal to $2(b_{st} t_{st}) + 12 t_w^2$; and for interior bearing, this effective area is equal to $2(b_{st} t_{st}) + 25 t_w^2$. t_w is the web thickness. The slenderness parameter, λ_c, is to be calculated using a radius of gyration, $r = \sqrt{(I_{st}/A_{eff})}$, where $I_{st} = t_{st}(2b_{st} + t_w)^3/12$.

For the limit state of bearing

The bearing strength, ϕR_n, must exceed the required compression force acting on the stiffeners. ϕR_n is given by

$$\phi R_n \ge 0.75[1.8 F_y A_{pb}] \qquad (3.91)$$

where F_y is the yield stress and A_{pb} is the bearing area.

Intermediate Stiffeners

Intermediate stiffeners shall be provided if (1) the shear strength capacity is calculated based on tension field action, (2) the shear criterion is violated (i.e., when the V_u exceeds $\phi_v V_n$), or (3) the web slenderness h/t_w exceeds $418/\sqrt{F_{yw}}$. Intermediate stiffeners can be provided in pairs or on one side of the web only in the form of plates or angles. They should be welded to the compression flange and the web but they may be stopped short of the tension flange. The following requirements apply to the design of intermediate stiffeners:

Local Buckling

The width-thickness ratio of the stiffener must be proportioned so that Equation 3.90 is satisfied to prevent failure by local buckling.

Stiffener Area

The cross-section area of the stiffener must satisfy the following criterion:

$$A_{st} \geq \frac{F_{yw}}{F_y} \left[0.15 D h t_w (1 - C_v) \frac{V_u}{\phi_v V_n} - 18 t_w^2 \right] \geq 0 \qquad (3.92)$$

where

F_y = yield stress of stiffeners

D = 1.0 for stiffeners in pairs, 1.8 for single angle stiffeners, and 2.4 for single plate stiffeners

The other terms in Equation 3.92 are defined as before in Equation 3.87 and Equation 3.88.

Stiffener Moment of Inertia

The moment of inertia for stiffener pairs taken about an axis in the web center or for single stiffeners taken in the face of contact with the web plate must satisfy the following criterion:

$$I_{st} \geq a t_w^3 \left[\frac{2.5}{(a/h)^2} - 2 \right] \geq 0.5 a t_w^3 \qquad (3.93)$$

Stiffener Length

The length of the stiffeners, l_{st}, should fall within the range

$$h - 6t_w < l_{st} < h - 4t_w \qquad (3.94)$$

where h is the clear distance between the flanges less the widths of the flange-to-web welds and t_w is the web thickness.

If intermittent welds are used to connect the stiffeners to the girder web, the clear distance between welds shall not exceed $16t_w$, or 10 in. If bolts are used, their spacing shall not exceed 12 in.

Stiffener Spacing

The spacing of the stiffeners, a, shall be determined from the shear criterion $\phi_v V_n \geq V_u$. This spacing shall not exceed the smaller of $3h$ and $[260/(h/t_w)]^2 h$.

EXAMPLE 3.7:

Using LRFD, design the cross-section of an I-shaped plate girder shown in Figure 3.12a to support a factored moment M_u of 4600 kip-ft (6240 kN-m), dead weight of the girder is included. The girder is a 60-ft (18.3-m) long simply supported girder. It is laterally supported at every 20-ft (6.1-m) interval. Use A36 steel.
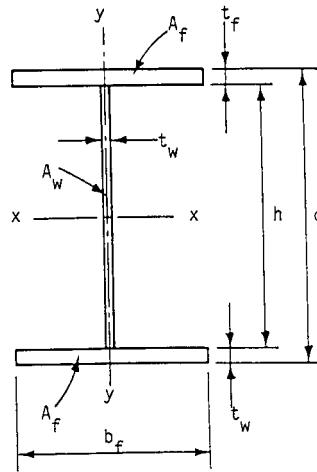
(a) Plate Girder Nomenclature
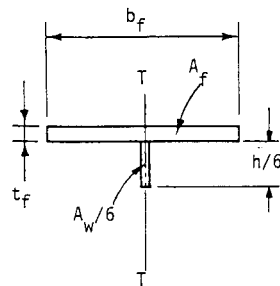
(b) Calculation of r_T

FIGURE 3.12: Design of a plate girder cross-section.

Proportion of the girder web

Ordinarily, the overall depth-to-span ratio d/L of a building girder is in the range 1/12 to 1/10. So, let us try $h = 70$ in. Also, knowing h/t_w of a plate girder is in the range $970/\sqrt{F_{yf}}$ and $2,000/\sqrt{F_{yf}}$, let us try $t_w = 5/16$ in.

Proportion of the girder flanges

For a preliminary design, the required area of the flange can be determined using the flange area method

$$A_f \approx \frac{M_u}{F_y h} = \frac{4600 \text{ kip-ft } x\,12 \text{ in./ft}}{(36 \text{ ksi })(70 \text{ in.})} = 21.7 \text{ in.}^2$$

So, let $b_f = 20$ in. and $t_f = 1\text{-}1/8$ in. giving $A_f = 22.5$ in.^{2}

Determine the design flexural strength $\phi_b M_n$ of the girder:
Calculate I_x:

$$
\begin{aligned}
I_x &= \sum [I_i + A_i y_i^2] \\
&= [8932 + (21.88)(0)^2] + 2[2.37 + (22.5)(35.56)^2] \\
&= 65840 \text{ in.}^4
\end{aligned}
$$

Calculate S_{xt}, S_{xc}:

$$
S_{xt} = S_{xc} = \frac{I_x}{c_t} = \frac{I_x}{c_c} = \frac{65840}{35 + 1.125} = 1823 \text{ in.}^3
$$

Calculate r_T : Refer to Figure 3.12b,

$$
r_T = \sqrt{\frac{I_T}{A_f + \frac{1}{6} A_w}} = \sqrt{\frac{(1.125)(20)^3/12 + (11.667)(5/16)^3/12}{22.5 + \frac{1}{6}(21.88)}} = 5.36 \text{ in.}
$$

Calculate F_{cr}:

For Flange Local Buckling (FLB),

$$
\left[\frac{b_f}{2t_f} = \frac{20}{2(1.125)} = 8.89 \right] < \left[\frac{65}{\sqrt{F_{yf}}} = \frac{65}{\sqrt{36}} = 10.8 \right] \quad \text{so,} \quad F_{cr} = F_{yf} = 36 \text{ ksi}
$$

For Lateral Torsional Buckling (LTB),

$$
\left[\frac{L_b}{r_T} = \frac{20 \times 12}{5.36} = 44.8 \right] < \left[\frac{300}{\sqrt{F_{yf}}} = \frac{300}{\sqrt{36}} = 50 \right] \quad \text{so,} \quad F_{cr} = F_{yf} = 36 \text{ ksi}
$$

Calculate R_{PG}:

$$
R_{PG} = 1 - \frac{a_r (h_c/t_w - 970/\sqrt{F_{cr}})}{(1,200 + 300 a_r)} = 1 - \frac{0.972[70/(5/16) - 970/\sqrt{36}]}{[1,200 + 300(0.972)]} = 0.96
$$

Calculate $\phi_b M_n$:

$$
\begin{aligned}
\phi_b M_n &= \text{smaller of} \begin{cases} 0.90 \, S_{xt} R_e F_y t = (0.90)(1823)(1)(36) = 59{,}065 \text{ kip-in.} \\ 0.90 \, S_{xc} R_{PG} R_e F_{cr} = (0.90)(1823)(0.96)(1)(36) = 56{,}700 \text{ kip-in.} \end{cases} \\
&= 56{,}700 \text{ kip-in.} \\
&= 4725 \text{ kip-ft.}
\end{aligned}
$$

Since [$\phi_b M_n = 4725$ kip-ft] > [$M_u = 4600$ kip-ft], the cross-section is acceptable.
Use web plate 5/16"x70" and two flange plates 1-1/8"x20" for the girder cross-section.

EXAMPLE 3.8:

Design bearing stiffeners for the plate girder of the preceding example for a factored end reaction of 260 kips.

Since the girder end is unframed, bearing stiffeners are required at the supports. The size of the stiffeners must be selected to ensure that the limit states of local buckling, compression, and bearing are not violated.

Limit state of local buckling

Refer to Figure 3.13, try $b_{st} = 8$ in. To avoid problems with local buckling, $b_{st}/2t_{st}$ must not exceed $95/\sqrt{F_y} = 15.8$. Therefore, try $t_{st} = 1/2$ in. So, $b_{st}/2t_{st} = 8$ which is less than 15.8.
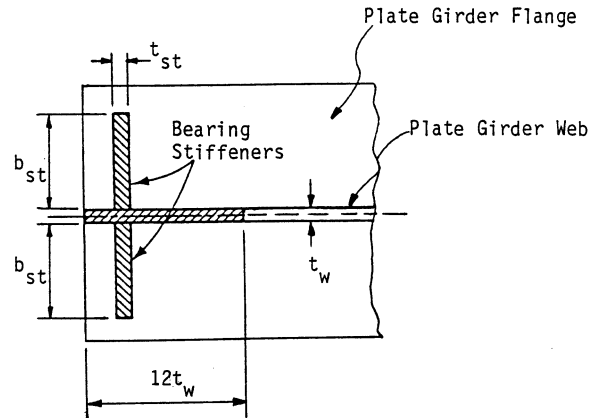

FIGURE 3.13: Design of bearing stiffeners.

Limit state of compression

$$
\begin{aligned}
A_{eff} &= 2(b_{st}t_{st}) + 12t_w^2 = 2(8)(0.5) + 12(5/16)^2 = 9.17 \text{ in.}^2 \\
I_{st} &= t_{st}(2b_{st} + t_w)^3/12 = 0.5[2(8) + 5/16]^3/12 = 181 \text{ in.}^4 \\
r_{st} &= \sqrt{(I_{st}/A_{eff})} = \sqrt{(181/9.17)} = 4.44 \text{ in.} \\
Kh/r_{st} &= 0.75(70)/4.44 = 11.8 \\
\lambda_c &= (Kh/\pi r_{st})\sqrt{(F_y/E)} = (11.8/3.142)\sqrt{(36/29,000)} = 0.132
\end{aligned}
$$

and from Equation 3.17

$$
\phi_c P_n = 0.85(0.658^{\lambda_c^2})F_y A_{st} = 0.85(0.658)^{0.132^2}(36)(9.17) = 279 \text{ kips}
$$

Since $\phi_c P_n > 260$ kips, the design is satisfactory for compression.

Limit state of bearing

Assuming there is a 1/4-in. weld cutout at the corners of the bearing stiffeners at the junction of the stiffeners and the girder flanges, the bearing area for the stiffener pairs is $A_{pb} = (8-0.25)(0.5)(2) = 7.75$ in.^{2}. Substitute this into Equation 3.91, we have $\phi R_n = 0.75(1.8)(36)(7.75) = 377$ kips, which exceeds the factored reaction of 260 kips. So, bearing is not a problem.
Use two 1/2"x 8" plates for bearing stiffeners.

3.11 Connections

Connections are structural elements used for joining different members of a framework. Connections can be classified according to:

- the type of connecting medium used: bolted connections, welded connections, bolted-welded connections, riveted connections
- the type of internal forces the connections are expected to transmit: shear (semi-rigid, simple) connections, moment (rigid) connections
- the type of structural elements that made up the connections: single plate angle connections, double web angle connections, top and seated angle connections, seated beam connections, etc.
- the type of members the connections are joining: beam-to-beam connections (beam splices), column-to-column connections (column splices), beam-to-column connections, hanger connections, etc.

To properly design a connection, a designer must have a thorough understanding of the behavior of the joint under loads. Different modes of failure can occur depending on the geometry of the connection and the relative strengths and stiffnesses of the various components of the connection. To ensure that the connection can carry the applied loads, a designer must check for all perceivable modes of failure pertinent to each component of the connection and the connection as a whole.

3.11.1   Bolted Connections

Bolted connections are connections whose components are fastened together primarily by bolts. The four basic types of bolts commonly used for steel construction are discussed in the section on Structural Fasteners. Depending on the direction and line of action of the loads relative to the orientation and location of the bolts, the bolts may be loaded in tension, shear, or a combination of tension and shear. For bolts subjected to shear forces, the design shear strength of the bolts also depends on whether or not the threads of the bolts are excluded from the shear planes. A letter X or N is placed at the end of the ASTM designation of the bolts to indicate whether the threads are excluded or not excluded from the shear planes, respectively. Thus, A325X denotes A325 bolts whose threads are excluded from the shear planes and A490N denotes A490 bolts whose threads are not excluded from the shear planes. Because of the reduced shear areas for bolts whose threads are not excluded from the shear planes, these bolts have lower design shear strengths than their counterparts whose threads are excluded from the shear planes.

Bolts can be used in both bearing-type connections and slip-critical connections. Bearing-type connections rely on bearing between the bolt shanks and the connecting parts to transmit forces. Some slippage between the connected parts is expected to occur for this type of connection. Slip-critical connections rely on the frictional force developing between the connecting parts to transmit forces. No slippage between connecting elements is expected for this type of connection. Slip-critical connections are used for structures designed for vibratory or dynamic loads such as bridges, industrial buildings, and buildings in regions of high seismicity. Bolts used in slip-critical connections are denoted by the letter F after their ASTM designation, e.g., A325F, A490F.

Bolt Holes

Holes made in the connected parts for bolts may be standard size, oversized, short slotted, or long slotted. Table 3.10 gives the maximum hole dimension for ordinary construction usage.

Standard holes can be used for both bearing-type and slip-critical connections. Oversized holes shall be used only for slip-critical connections. Short- and long-slotted holes can be used for both bearing-type and slip-critical connections provided that when such holes are used for bearing, the direction of slot is transverse to the direction of loading.

TABLE 3.10 Nominal Hole Dimensions

Bolt diameter, d (in.)	Standard (dia.)	Oversize (dia.)	Short-slot (width × length)	Long-slot (width × length)
			Hole dimensions	
1/2	9/16	5/8	9/16×11/16	9/16×1-1/4
5/8	11/16	13/16	11/16×7/8	11/16×1-9/16
3/4	13/16	15/16	13/16×1	13/16×1-7/8
7/8	15/16	1-1/16	15/16×1-1/8	15/16×2-3/16
1	1-1/16	1-1/4	1-1/16×1-5/16	1-1/16×2-1/2
≥ 1-1/8	d+1/16	d+5/16	(d+1/16)×(d+3/8)	(d+1/16)×(2.5d)

Note: 1 in. = 25.4 mm.

Bolts Loaded in Tension

If a tensile force is applied to the connection such that the direction of the load is parallel to the longitudinal axes of the bolts, the bolts will be subjected to tension. The following condition must be satisfied for bolts under tensile stresses.

Allowable Stress Design:

$$f_t \leq F_t \tag{3.95}$$

where

f_t = computed tensile stress in the bolt, ksi
F_t = allowable tensile stress in bolt (see Table 3.11)

Load and Resistance Factor Design:

$$\phi_t F_t \geq f_t \tag{3.96}$$

where

ϕ_t = 0.75
f_t = tensile stress produced by factored loads, ksi
F_t = nominal tensile strength given in Table 3.11

TABLE 3.11 F_t of Bolts, ksi

Bolt type	ASD		LRFD	
	F_t, ksi (static loading)	F_t, ksi (fatigue loading)	F_t, ksi (static loading)	F_t, ksi (fatigue loading)
A307	20	Not allowed	45.0	Not allowed
A325	44.0	If $N \leq 20{,}000$: F_t = same as for static loading	90.0	If $N \leq 20{,}000$: F_t = same as for static loading
		If $20{,}000 < N \leq 500{,}000$: $F_t = 40$ (A325) $= 49$ (A490)		If $20{,}000 < N \leq 500{,}000$: $F_t = 0.30F_u$ (at service loads)
		If $N > 500{,}000$:		
A490	54.0	$F_t = 31$ (A325) $= 38$ (A490)	113	If $N > 500{,}000$: $F_t = 0.25F_u$ (at service loads)
		where N = number of cycles F_u = minimum specified tensile strength, ksi		where N = number of cycles F_u = minimum specified tensile strength, ksi

Note: 1 ksi = 6.895 MPa.

Bolts Loaded in Shear

When the direction of load is perpendicular to the longitudinal axes of the bolts, the bolts will be subjected to shear. The condition that needs to be satisfied for bolts under shear stresses is as follows.

Allowable Stress Design:

$$f_v \leq F_v \tag{3.97}$$

where

f_v = computed shear stress in the bolt, ksi
F_v = allowable shear stress in bolt (see Table 3.12)

Load and Resistance Factor Design:

$$\phi_v F_v \geq f_v \tag{3.98}$$

where

ϕ_v = 0.75 (for bearing-type connections), 1.00 (for slip-critical connections when standard, over-sized, short-slotted, or long-slotted holes with load perpendicular to the slots are used), 0.85 (for slip-critical connections when long-slotted holes with load in the direction of the slots are used)
f_v = shear stress produced by factored loads (for bearing-type connections), or by service loads (for slip-critical connections), ksi
F_v = nominal shear strength given in Table 3.12

TABLE 3.12 F_v of Bolts, ksi

Bolt type	F_v, ksi	
	ASD	LRFD
A307	10.0[a] (regardless of whether or not threads are excluded from shear planes)	24.0[a] (regardless of whether or not threads are excluded from shear planes)
A325N	21.0[a]	48.0[a]
A325X	30.0[a]	60.0[a]
A325F[b]	17.0 (for standard size holes) 15.0 (for oversized and short-slotted holes) 12.0 (for long-slotted holes when direction of load is transverse to the slots) 10.0 (for long-slotted holes when direction of load is parallel to the slots)	17.0 (for standard size holes) 15.0 (for oversized and short-slotted holes) 12.0 (for long-slotted holes)
A490N	28.0[a]	60.0[a]
A490X	40.0[a]	75.0[a]
A490F[b]	21.0 (for standard size holes) 18.0 (for oversized and short-slotted holes) 15.0 (for long-slotted holes when direction of load is transverse to the slots) 13.0 (for long-slotted holes when direction of load is parallel to the slots)	21.0 (for standard size holes) 18.0 (for oversized and short-slotted holes) 15.0 (for long-slotted holes)

[a] tabulated values shall be reduced by 20% if the bolts are used to splice tension members having a fastener pattern whose length, measured parallel to the line of action of the force, exceeds 50 in.

[b] tabulated values are applicable only to class A surface, i.e., clean mill surface and blast cleaned surface with class A coatings (with slip coefficient = 0.33). For design strengths with other coatings, see RCSC "Load and Resistance Factor Design Specification to Structural Joints Using ASTM A325 or A490 Bolts" [28]

Note: 1 ksi = 6.895 MPa.

Bolts Loaded in Combined Tension and Shear

If a tensile force is applied to a connection such that its line of action is at an angle with the longitudinal axes of the bolts, the bolts will be subjected to combined tension and shear. The conditions that need to be satisfied are given as follows.

Allowable Stress Design:

$$f_v \leq F_v \quad \text{and} \quad f_t \leq F_t \tag{3.99}$$

where

f_v, F_v =   as defined in Equation 3.97
f_t =   computed tensile stress in the bolt, ksi
F_t =   allowable tensile stress given in Table 3.13

Load and Resistance Factor Design:

$$\phi_v F_v \geq f_v \quad \text{and} \quad \phi_t F_t \geq f_t \tag{3.100}$$

where

$\phi_v, F_v, f_v =$ as defined in Equation 3.98
ϕ_t =   1.0
f_t =   tensile stress due to factored loads (for bearing-type connection), or due to service loads (for slip-critical connections), ksi
F_t =   nominal tension stress limit for combined tension and shear given in Table 3.13

TABLE 3.13 F_t for Bolts Under Combined Tension and Shear, ksi

	Bearing-type connections			
	ASD		LRFD	
Bolt type	Threads not excluded from the shear plane	Threads excluded from the shear plane	Threads not excluded from the shear plane	Threads excluded from the shear plane
A307	$26\text{-}1.8 f_v \leq 20$		$59\text{-}1.9 f_v \leq 45$	
A325	$\sqrt{(44^2 - 4.39 f_v^2)}$	$\sqrt{(44^2 - 2.15 f_v^2)}$	$117 - 1.9 f_v \leq 90$	$117 - 1.5 f_v \leq 90$
A490	$\sqrt{(54^2 - 3.75 f_v^2)}$	$\sqrt{(54^2 - 1.82 f_v^2)}$	$147 - 1.9 f_v \leq 113$	$147 - 1.5 f_v \leq 113$

Slip-critical connections

For ASD:
F_t =   values given above
F_v = $[1 - (f_t A_b / T_b)] \times$ (values of F_v given in Table 3.12)

where
f_t =   computed tensile stress in the bolt, ksi
T_b =   pretension load $= 0.70 F_u A_b$, kips
F_u =   minimum specified tensile strength, ksi
A_b =   nominal cross-sectional area of bolt, in.^{2}

For LRFD:
F_t =   values given above
F_v = $[1 - (T / T_b)] \times$ (values of F_v given in Table 3.12)

where
T =   service tensile force, kips
T_b =   pretension load $= 0.70 F_u A_b$, kips
F_u =   minimum specified tensile strength, ksi
A_b =   nominal cross-sectional area of bolt, in.^{2}

Note: 1 ksi = 6.895 MPa.

Bearing Strength at Fastener Holes

Connections designed on the basis of bearing rely on the bearing force developed between the fasteners and the holes to transmit forces and moments. The limit state for bearing must therefore be checked to ensure that bearing failure will not occur. Bearing strength is independent of the type of fastener. This is because the bearing stress is more critical on the parts being connected than on the fastener itself. The AISC specification provisions for bearing strength are based on preventing

excessive hole deformation. As a result, bearing capacity is expressed as a function of the type of holes (standard, oversized, slotted), bearing area (bolt diameter times the thickness of the connected parts), bolt spacing, edge distance (L_e), strength of the connected parts (F_u) and the number of fasteners in the direction of the bearing force. Table 3.14 summarizes the expressions used in ASD and LRFD for calculating the bearing strength and the conditions under which each expression is valid.

TABLE 3.14   Bearing Capacity

Conditions	ASD Allowable bearing stress, F_p, ksi	LRFD Design bearing strength, ϕR_n, ksi
1. For standard or short-slotted holes with $L_e \geq 1.5d$, $s \geq 3d$ and number of fasteners in the direction of bearing ≥ 2	$1.2F_u$	$0.75[2.4dt F_u]$
2. For long-slotted holes with direction of slot transverse to the direction of bearing and $L_e \geq 1.5d$, $s \geq 3d$ and the number of fasteners in the direction of bearing ≥ 2	$1.0F_u$	$0.75[2.0dt F_u]$
3. If neither condition 1 nor 2 above is satisfied	$L_e F_u/2d \leq 1.2F_u$	For the bolt hole nearest the edge: $0.75[L_e t F_u]$ $\leq 0.75[2.4dt F_u]^a$ For the remaining bolt holes: $0.75[(s - d/2)t F_u]$ $\leq 0.75[2.4dt F_u]^a$
4. If hole deformation is not a design consideration and adequate spacing and edge distance is provided (see sections on Minimum Fastener Spacing and Minimum Edge Distance)	$1.5F_u$	For the bolt hole nearest the edge: $0.75[L_e t F_u]$ $\leq 0.75[3.0dt F_u]$ For the remaining bolt holes: $0.75[(s - d/2)t F_u]$ $\leq 0.75[3.0dt F_u]$

[a] For long-slotted bolt holes with direction of slot transverse to the direction of bearing, this limit is $0.75[2.0dt F_u]$

L_e = edge distance (i.e., distance measured from the edge of the connected part to the center of a standard hole or the center of a short- and long-slotted hole perpendicular to the line of force. For oversized holes and short- and long-slotted holes parallel to the line of force, L_e shall be increased by the edge distance increment C_2 given in Table 3.16)

s = fastener spacing (i.e., center to center distance between adjacent fasteners measured in the direction of bearing. For oversized holes and short- and long-slotted holes parallel to the line of force, s shall be increased by the spacing increment C_1 given in Table 3.15)

d = nominal bolt diameter, in.

t = thickness of the connected part, in.

F_u = specified minimum tensile strength of the connected part, ksi

TABLE 3.15   Values of Spacing Increment, C_1, in.

Nominal diameter of fastener (in.)	Standard holes	Oversized holes	Slotted Holes Transverse to line of force	Parallel to line of force Short-slots	Long-slots[a]
$\leq 7/8$	0	1/8	0	3/16	$3d/2$-1/16
1	0	3/16	0	1/4	23/16
≥ 1-1/8	0	1/4	0	5/16	$3d/2$-1/16

[a] When length of slot is less than the value shown in Table 3.10, C_1 may be reduced by the difference between the value shown and the actual slot length.

Note: 1 in. = 25.4 mm.

Minimum Fastener Spacing

To ensure safety, efficiency, and to maintain clearances between bolt nuts as well as to provide room for wrench sockets, the fastener spacing, s, should not be less than $3d$ where d is the nominal fastener diameter.

TABLE 3.16 Values of Edge Distance Increment, C_2, in.

| Nominal diameter of fastener (in.) | Oversized holes | Slotted holes | | Slot parallel to edge |
| | | Slot transverse to edge | | |
		Short-slot	Long-slot[a]	
$\leq 7/8$	1/16	1/8	$3d/4$	0
1	1/8	1/8	$3d/4$	
$\leq 1\text{-}1/8$	1/8	3/16	$3d/4$	

[a] If the length of the slot is less than the maximum shown in Table 3.10, the value shown may be reduced by one-half the difference between the maximum and the actual slot lengths.

Note: 1 in. = 25.4 mm.

Minimum Edge Distance

To prevent excessive deformation and shear rupture at the edge of the connected part, a minimum edge distance L_e must be provided in accordance with the values given in Table 3.17 for standard holes. For oversized and slotted holes, the values shown must be incremented by C_2 given in Table 3.16.

TABLE 3.17 Minimum Edge Distance for Standard Holes, in.

Nominal fastener diameter (in.)	At sheared edges	At rolled edges of plates, shapes, and bars or gas cut edges
1/2	7/8	3/4
5/8	1-1/8	7/8
3/4	1-1/4	1
7/8	1-1/2	1-1/8
1	1-3/4	1-1/4
1-1/8	2	1-1/2
1-1/4	2-1/4	1-5/8
over 1-1/4	1-3/4 x diameter	1-1/4 x diameter

Note: 1 in. = 25.4 mm.

Maximum Fastener Spacing

A limit is placed on the maximum value for the spacing between adjacent fasteners to prevent the possibility of gaps forming or buckling from occurring in between fasteners when the load to be transmitted by the connection is compressive. The maximum fastener spacing measured in the direction of the force is given as follows.

For painted members or unpainted members not subject to corrosion: smaller of $24t$ where t is the thickness of the thinner plate and 12 in.

For unpainted members of weathering steel subject to atmospheric corrosion: smaller of $14t$ where t is the thickness of the thinner plate and 7 in.

Maximum Edge Distance

A limit is placed on the maximum value for edge distance to prevent prying action from occurring. The maximum edge distance shall not exceed the smaller of 12_t where t is the thickness of the connected part and 6 in.

EXAMPLE 3.9:

Check the adequacy of the connection shown in Figure 3.4a. The bolts are 1-in. diameter A325N bolts in standard holes.

Check bolt capacity

All bolts are subjected to double shear. Therefore, the design shear strength of the bolts will be twice that shown in Table 3.12. Assuming each bolt carries an equal share of the factored applied load, we have from Equation 3.98

$$[\phi_v F_v = 0.75(2 \times 48) = 72 \ \text{ksi}] > \left[f_v = \frac{208}{(6)\left(\frac{\pi 1^2}{4}\right)} = 44.1 \ \text{ksi} \right]$$

The shear capacity of the bolt is therefore adequate.

Check bearing capacity of the connected parts

With reference to Table 3.14, it can be seen that condition 1 applies for the present problem. Therefore, we have

$$\left[\phi R_n = 0.75(2.4)(1)\left(\frac{3}{8}\right)(58) = 39.2 \ \text{kips}\right] > \left[R_u = \frac{208}{6} = 34.7 \ \text{kips}\right]$$

and so bearing is not a problem. Note that bearing on the gusset plate is more critical than bearing on the webs of the channels because the thickness of the gusset plate is less than the combined thickness of the double channels.

Check bolt spacing

The minimum bolt spacing is $3d = 3(1) = 3$ in. The maximum bolt spacing is the smaller of $14t = 14(.303) = 4.24$ in. or 7 in. The actual spacing is 3 in. which falls within the range of 3 to 4.24 in., so bolt spacing is adequate.

Check edge distance

From Table 3.17, it can be determined that the minimum edge distance is 1.25 in. The maximum edge distance allowed is the smaller of $12t = 12(0.303) = 3.64$ in. or 6 in. The actual edge distance is 3 in. which falls within the range of 1.25 to 3.64 in., so edge distance is adequate.

The connection is adequate.

Bolted Hanger Type Connections

A typical hanger connection is shown in Figure 3.14. In the design of such connections, the designer must take into account the effect of *prying action*. Prying action results when flexural deformation occurs in the tee flange or angle leg of the connection (Figure 3.15). Prying action tends to increase the tensile force, called prying force, in the bolts. To minimize the effect of prying, the fasteners should be placed as close to the tee stem or outstanding angle leg as the wrench clearance will permit (see Tables on Entering and Tightening Clearances in Volume II-Connections of the AISC-LRFD Manual [22]). In addition, the flange and angle thickness should be proportioned so that the full tensile capacities of the bolts can be developed.
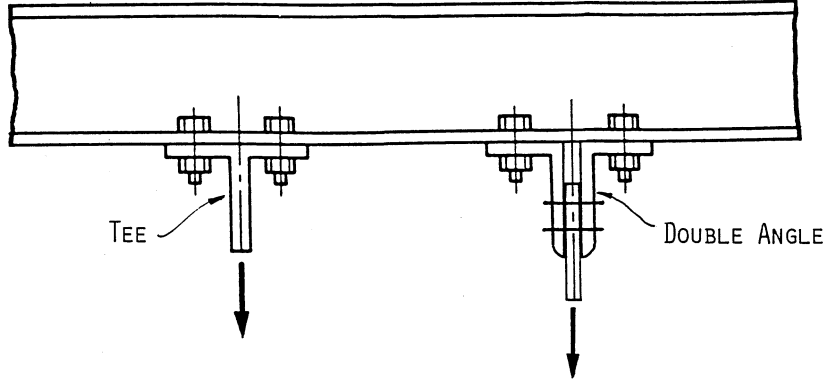
FIGURE 3.14: Hanger connections.

Two failure modes can be identified for hanger type connections: formation of plastic hinges in the tee flange or angle leg at cross-sections 1 and 2, and tensile failure of the bolts when the tensile force including prying action $B_c(= T + Q)$ exceeds the tensile capacity of the bolt B. Since the determination of the actual prying force is rather complex, the design equation for the required thickness for the tee flange or angle leg is semi-empirical in nature. It is given by the following.

If ASD is used:

$$t_{req'd} = \sqrt{\frac{8Tb'}{pF_y(1 + \delta\alpha')}} \tag{3.101}$$

where
T = tensile force per bolt due to service load exclusive of initial tightening and prying force, kips

The other variables are as defined in Equation 3.102 except that B in the equation for α' is defined as the allowable tensile force per bolt. A design is considered satisfactory if the thickness of the tee flange or angle leg t_f exceeds $t_{req'd}$ and $B > T$.

If LRFD is used:

$$t_{req'd} = \sqrt{\frac{4T_u b'}{\phi_b pF_y(1 + \delta\alpha')}} \tag{3.102}$$

where
ϕ_b = 0.90
T_u = factored tensile force per bolt exclusive of initial tightening and prying force, kips
p = length of flange tributary to each bolt measured along the longitudinal axis of the tee or double angle section, in.
δ = ratio of net area at bolt line to gross area at angle leg or stem face = $(p - d')/p$
d' = diameter of bolt hole = bolt diameter $+1/8''$, in.
α' = $[(B/T_u - 1)(a'/b')]/\{\delta[1 - (B/T_u - 1)(a'/b')]\}$, but not larger than 1 (if α' is less than zero, use $\alpha' = 1$)
B = design tensile strength of one bolt = $\phi F_t A_b$, kips (ϕF_t is given in Table 3.11 and A_b is the nominal diameter of the bolt)
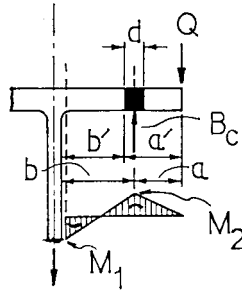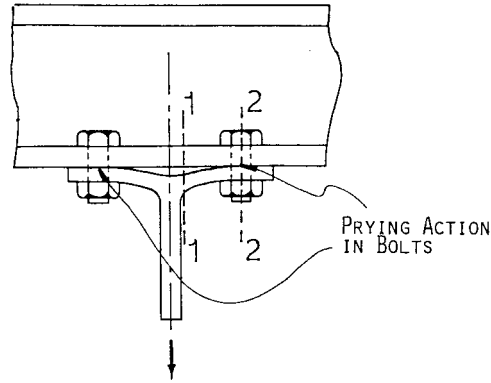a' = $a + d/2$
b' = $b - d/2$

FIGURE 3.15: Prying action in hanger connections.

a = distance from bolt centerline to edge of tee flange or angle leg but not more than $1.25b$, in.
b = distance from bolt centerline to face of tee stem or outstanding leg, in.

A design is considered satisfactory if the thickness of the tee flange or angle leg t_f exceeds $t_{reg'd}$ and $B > T_u$.

Note that if t_f is much larger than $t_{reg'd}$, the design will be too conservative. In this case α' should be recomputed using the equation

$$\alpha' = \frac{1}{\delta}\left[\frac{4T_u b'}{\phi_b p t_f^2 F_y} - 1\right] \qquad (3.103)$$

As before, the value of α' should be limited to the range $0 \le \alpha' \le 1$. This new value of α' is to be used in Equation 3.102 to recalculate $t_{reg'd}$.

Bolted Bracket Type Connections

Figure 3.16 shows three commonly used bracket type connections. The bracing connection shown in Figure 3.16a should be designed so that the line of action the force passes through is the centroid of the bolt group. It is apparent that the bolts connecting the bracket to the column flange are subjected to combined tension and shear. As a result, the capacity of the connection is limited
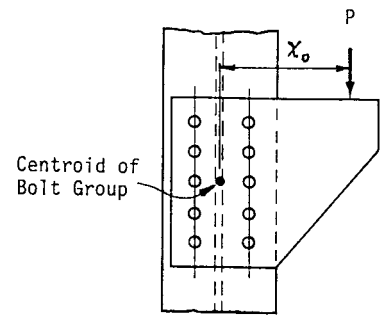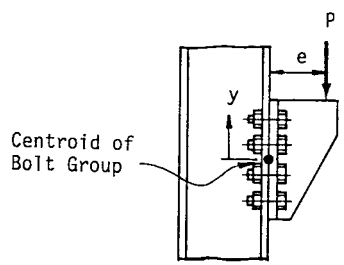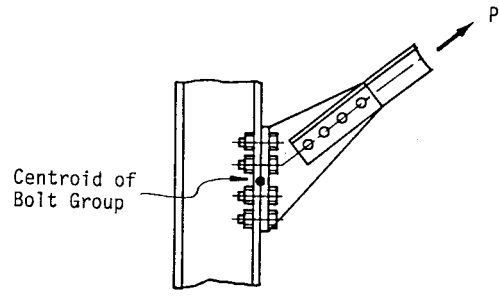
FIGURE 3.16: Bolted bracket-type connections.

to the combined tensile-shear capacities of the bolts in accordance with Equation 3.99 in ASD and Equation 3.100 in LRFD. For simplicity, f_v and f_t are to be computed assuming that both the tensile and shear components of the force are distributed evenly to all bolts. In addition to checking for the bolt capacities, the bearing capacities of the column flange and the bracket should also be checked. If the axial component of the force is significant, the effect of prying should also be considered.

In the design of the eccentrically loaded connections shown in Figure 3.16b, it is assumed that the neutral axis of the connection lies at the center of gravity of the bolt group. As a result, the bolts above the neutral axis will be subjected to combined tension and shear and so Equation 3.99 or Equation 3.100 needs to be checked. The bolts below the neutral axis are subjected to shear only and so Equation 3.97 or Equation 3.98 applies. In calculating f_v, one can assume that all bolts in the bolt group carry an equal share of the shear force. In calculating f_t, one can assume that the tensile force varies linearly from a value of zero at the neutral axis to a maximum value at the bolt farthest away from the neutral axis. Using this assumption, f_t can be calculated from the equation Pey/I where y is the distance from the neutral axis to the location of the bolt above the neutral axis and $I = \sum A_b y^2$ is the moment of inertia of the bolt areas with A_b equal to the cross-sectional area of each bolt. The capacity of the connection is determined by the capacities of the bolts and the bearing capacity of the connected parts.

For the eccentrically loaded bracket connection shown in Figure 3.16c, the bolts are subjected to shear. The shear force in each bolt can be obtained by adding vectorally the shear caused by the applied load P and the moment $P\chi_o$. The design of this type of connection is facilitated by the use of tables contained in the AISC Manuals for Allowable Stress Design and Load and Resistance Factor Design [21, 22].

In addition to checking for bolt shear capacity, one needs to check the bearing and shear rupture capacities of the bracket plate to ensure that failure will not occur in the plate.

Bolted Shear Connections

Shear connections are connections designed to resist shear force only. These connections are not expected to provide appreciable moment restraint to the connection members. Examples of these connections are shown in Figure 3.17. The framed beam connection shown in Figure 3.17a consists of two web angles which are often shop-bolted to the beam web and then field-bolted to the column flange. The seated beam connection shown in Figure 3.17b consists of two flange angles often shop-bolted to the beam flange and field-bolted to the column flange. To enhance the strength and stiffness of the seated beam connection, a stiffened seated beam connection shown in Figure 3.17c is sometimes used to resist large shear force. Shear connections must be designed to sustain appreciable deformation and yielding of the connections is expected. The need for ductility often limits the thickness of the angles that can be used. Most of these connections are designed with angle thickness not exceeding 5/8 in.

The design of the connections shown in Figure 3.17 is facilitated by the use of design tables contained in the AISC-ASD and AISC-LRFD Manuals. These tables give design loads for the connections with specific dimensions based on the limit states of bolt shear, bearing strength of the connection, bolt bearing with different edge distances, and block shear (for coped beams).

Bolted Moment-Resisting Connections

Moment-resisting connections are connections designed to resist both moment and shear. These connections are often referred to as rigid or fully restrained connections as they provide full continuity between the connected members and are designed to carry the full factored moments. Figure 3.18 shows some examples of moment-resisting connections. Additional examples can be found in the AISC-ASD and AISC-LRFD Manuals and Chapter 4 of the AISC Manual on Connections [20].

(a) Bolted Framed Beam Connection

Use shims as required

(b) Bolted Seated Beam Connection

Use shims as required

Stiffeners fitted to bear

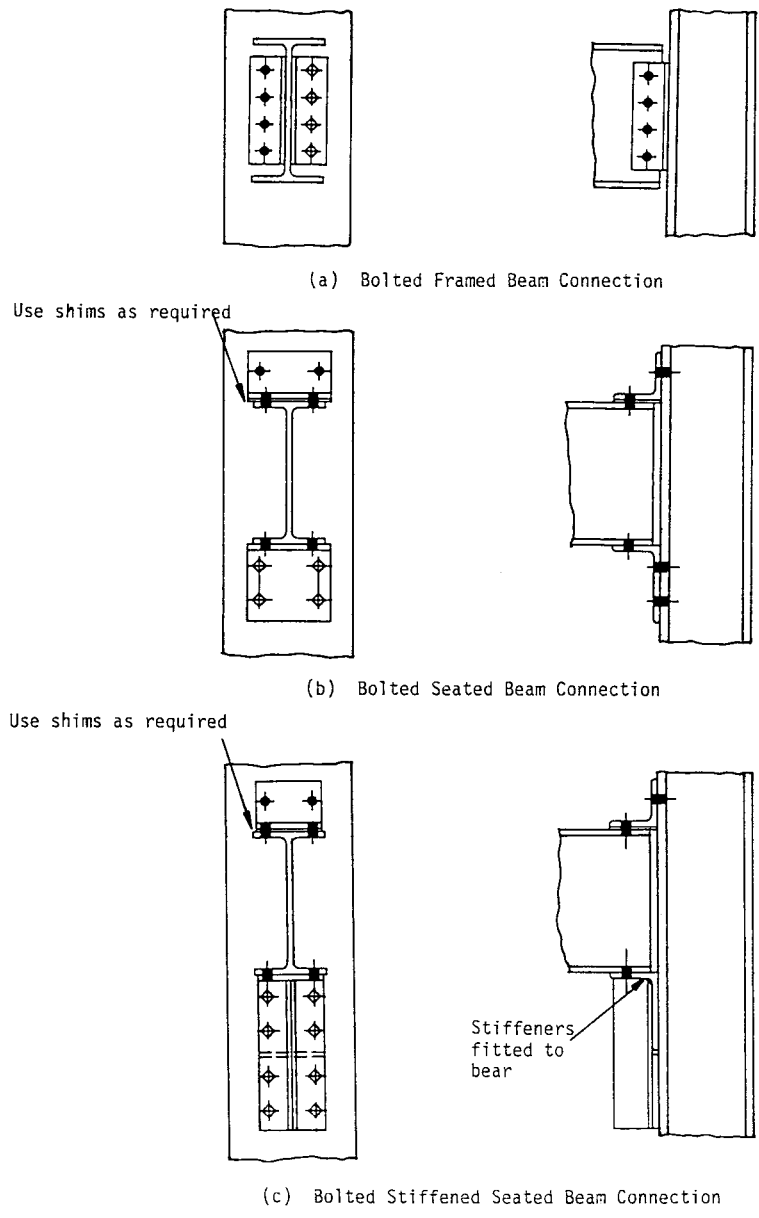(c) Bolted Stiffened Seated Beam Connection

FIGURE 3.17: Bolted shear connections. (a) Bolted frame beam connection. (b) Bolted seated beam connection. (c) Bolted stiffened seated beam connection.
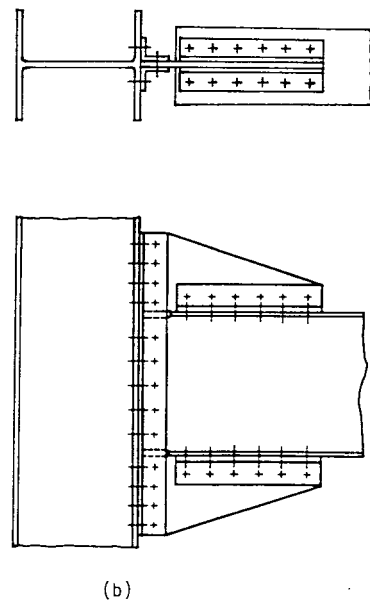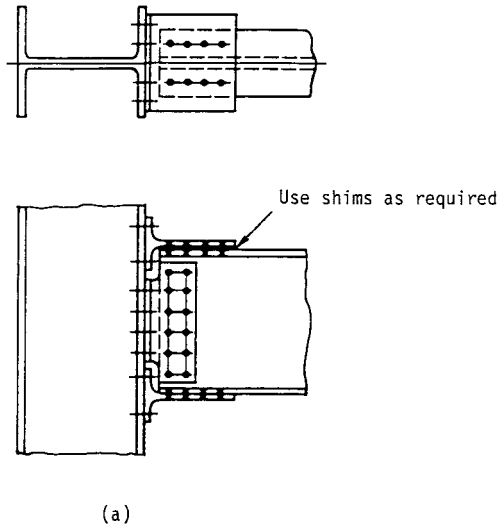
Use shims as required

(a)

(b)

FIGURE 3.18: Bolted moment connections.

Design of Moment-Resisting Connections

An assumption used quite often in the design of moment connections is that the moment is carried solely by the flanges of the beam. The moment is converted to a couple F_f given by $F_f = M/(d - t_f)$ acting on the beam flanges as shown in Figure 3.19.
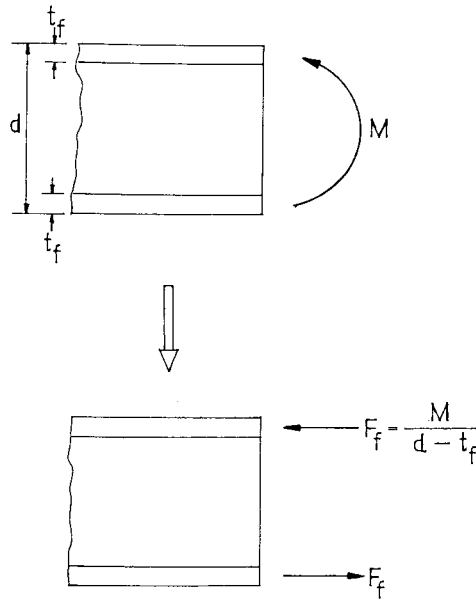

FIGURE 3.19: Flange forces in moment connections.

The design of the connection for moment is considered satisfactory if the capacities of the bolts and connecting plates or structural elements are adequate to carry the flange force F_f. Depending on the geometry of the bolted connection, this may involve checking: (a) the shear and/or tensile capacities of the bolts, (b) the yield and/or fracture strength of the moment plate, (c) the bearing strength of the connected parts, and (d) bolt spacing and edge distance as discussed in the foregoing sections.

As for shear, it is common practice to assume that all the shear resistance is provided by the shear plates or angles. The design of the shear plates or angles is governed by the limit states of bolt shear, bearing of the connected parts, and shear rupture.

If the moment to be resisted is large, the flange force may cause bending of the column flange, or local yielding, crippling, or buckling of the column web. To prevent failure due to bending of the column flange or local yielding of the column web (for a tensile F_f) as well as local yielding, crippling or buckling of the column web (for a compressive F_f), column stiffeners should be provided if any one of the conditions discussed in the section on Criteria on Concentrated Loads is violated.

Following is a set of guidelines for the design of column web stiffeners [21, 22]:

1. If local web yielding controls, the area of the stiffeners (provided in pairs) shall be determined based on any excess force beyond that which can be resisted by the web alone. The stiffeners need not extend more than one-half the depth of the column web if the concentrated beam flange force F_f is applied at only one column flange.

2. If web crippling or compression buckling of the web controls, the stiffeners shall be designed as axially loaded compression members (see section on Compression Members). The stiffeners shall extend the entire depth of the column web.

3. The welds that connect the stiffeners to the column shall be designed to develop the full strength of the stiffeners.

In addition, the following recommendations are given:

1. The width of the stiffener plus one-half of the column web thickness should not be less than one-half the width of the beam flange nor the moment connection plate which applies the force.

2. The stiffener thickness should not be less than one-half the thickness of the beam flange.

3. If only one flange of the column is connected by a moment connection, the length of the stiffener plate does not have to exceed one-half the column depth.

4. If both flanges of the column are connected by moment connections, the stiffener plate should extend through the depth of the column web and welds should be used to connect the stiffener plate to the column web with sufficient strength to carry the unbalanced moment on opposite sides of the column.

5. If column stiffeners are required on both the tension and compression sides of the beam, the size of the stiffeners on the tension side of the beam should be equal to that on the compression size for ease of construction.

In lieu of stiffener plates, a stronger column section could be used to preclude failure in the column flange and web.

For a more thorough discussion of bolted connections, the readers are referred to the book by Kulak et al. [16]. Examples on the design of a variety of bolted connections can be found in the AISC-LRFD Manual [22] and the AISC Manual on Connections [20]

3.11.2   Welded Connections

Welded connections are connections whose components are joined together primarily by welds. The four most commonly used welding processes are discussed in the section on Structural Fasteners. Welds can be classified according to:

- types of welds: groove, fillet, plug, and slot welds.
- positions of the welds: horizontal, vertical, overhead, and flat welds.
- types of joints: butt, lap, corner, edge, and tee.

Although fillet welds are generally weaker than groove welds, they are used more often because they allow for larger tolerances during erection than groove welds. Plug and slot welds are expensive to make and they do not provide much reliability in transmitting tensile forces perpendicular to the faying surfaces. Furthermore, quality control of such welds is difficult because inspection of the welds is rather arduous. As a result, plug and slot welds are normally used just for stitching different parts of the members together.

Welding Symbols

A shorthand notation giving important information on the location, size, length, etc. for the various types of welds was developed by the American Welding Society [6] to facilitate the detailing of welds. This system of notation is reproduced in Figure 3.20.
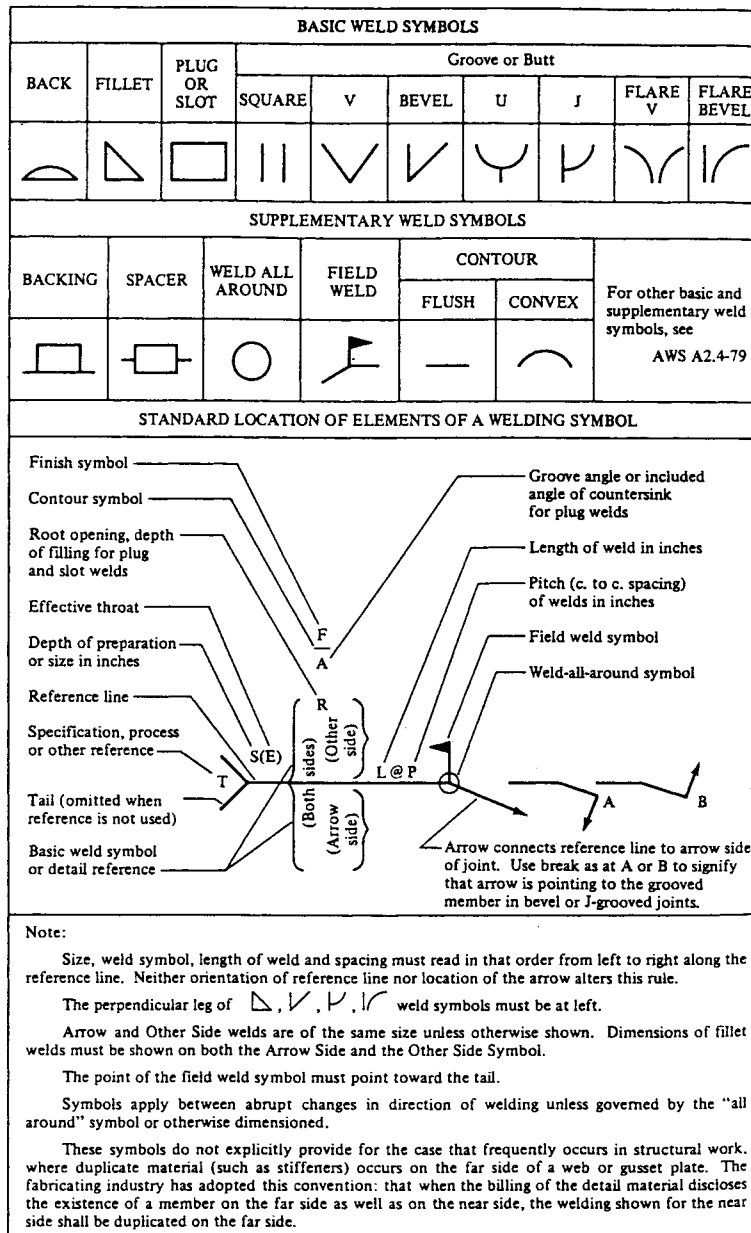
BASIC WELD SYMBOLS

BACK	FILLET	PLUG OR SLOT	Groove or Butt						
			SQUARE	V	BEVEL	U	J	FLARE V	FLARE BEVEL
⌒	◺	☐	‖	∨	�	⊻	⊬	)(	)⌐

SUPPLEMENTARY WELD SYMBOLS

BACKING	SPACER	WELD ALL AROUND	FIELD WELD	CONTOUR		For other basic and supplementary weld symbols, see AWS A2.4-79
				FLUSH	CONVEX	
⊓	⊟	◯	⌐	—	⌒	

STANDARD LOCATION OF ELEMENTS OF A WELDING SYMBOL

Finish symbol

Contour symbol

Root opening, depth of filling for plug and slot welds

Effective throat

Depth of preparation or size in inches

Reference line

Specification, process or other reference

Tail (omitted when reference is not used)

Basic weld symbol or detail reference

F
A
R
S(E)
T
(Other side)
(Both sides)
(Arrow side)
L @ P

Groove angle or included angle of countersink for plug welds

Length of weld in inches

Pitch (c. to c. spacing) of welds in inches

Field weld symbol

Weld-all-around symbol

A   B

Arrow connects reference line to arrow side of joint. Use break as at A or B to signify that arrow is pointing to the grooved member in bevel or J-grooved joints.

Note:

Size, weld symbol, length of weld and spacing must read in that order from left to right along the reference line. Neither orientation of reference line nor location of the arrow alters this rule.

The perpendicular leg of ◺, V, ⊬,)⌐ weld symbols must be at left.

Arrow and Other Side welds are of the same size unless otherwise shown. Dimensions of fillet welds must be shown on both the Arrow Side and the Other Side Symbol.

The point of the field weld symbol must point toward the tail.

Symbols apply between abrupt changes in direction of welding unless governed by the "all around" symbol or otherwise dimensioned.

These symbols do not explicitly provide for the case that frequently occurs in structural work. where duplicate material (such as stiffeners) occurs on the far side of a web or gusset plate. The fabricating industry has adopted this convention: that when the billing of the detail material discloses the existence of a member on the far side as well as on the near side, the welding shown for the near side shall be duplicated on the far side.

FIGURE 3.20: Basic weld symbols.

Strength of Welds

In ASD, the strength of welds is expressed in terms of allowable stress. In LRFD, the design strength of welds is taken as the smaller of the design strength of the base material ϕF_{BM} and the design strength of the weld electrode ϕF_W. These allowable stresses and design strengths are summarized in Table 3.18 [18, 21]. When a design uses ASD, the computed stress in the weld shall not exceed its allowable value. When a design uses LRFD, the design strength of welds should exceed the required strength obtained by dividing the load to be transmitted by the effective area of the welds.

TABLE 3.18   Strength of Welds

Types of weld and stress[a]	Material	ASD allowable stress	LRFD ϕF_{BM} or ϕF_W	Required weld strength level[b,c]
Full penetration groove weld				
Tension normal to effective area	Base	Same as base metal	$0.90F_y$	"Matching" weld must be used
Compression normal to effective area	Base	Same as base metal	$0.90F_y$	Weld metal with a strength level equal to
Tension of compression parallel to axis of weld	Base	Same as base metal	$0.90F_y$	or less than "matching" must be used
Shear on effective area	Base weld electrode	$0.30\times$ nominal tensile strength of weld metal	$0.90[0.60F_y]$ $0.80[0.60F_{EXX}]$	
Partial penetration groove welds				
Compression normal to effective area	Base	Same as base metal	$0.90F_y$	Weld metal with a strength level equal to
Tension or compression parallel to axis of weld[d]				or less than "matching" weld metal may be used
Shear parallel to axis of weld	Base weld electrode	$0.30\times$ nominal tensile strength of weld metal	$0.75[0.60F_{EXX}]$	
Tension normal to effective area	Base weld electrode	$0.30\times$ nominal tensile strength of weld metal $\leq 0.18\times$ yield stress of base metal	$0.90F_y$ $0.80[0.60F_{EXX}]$	
Fillet welds				
Stress on effective area	Base weld electrode	$0.30\times$ nominal tensile strength of weld metal	$0.75[0.60F_{EXX}]$ $0.90F_y$	Weld metal with a strength level equal to or less than "matching" weld metal may be used
Tension or compression parallel to axis of weld[d]	Base	Same as base metal	$0.90F_y$	
Plug or slot welds				
Shear parallel to faying surfaces (on effective area)	Base weld electrode	$0.30\times$nominal tensile strength of weld metal	$0.75[0.60F_{EXX}]$	Weld metal with a strength level equal to or less than "matching" weld metal may be used

[a] see below for effective area
[b] see AWS D1.1 for "matching" weld material
[c] weld metal one strength level stronger than "matching" weld metal will be permitted
[d] fillet welds partial-penetration groove welds joining component elements of built-up members such as flange-to-web connections may be designed without regard to the tensile or compressive stress in these elements parallel to the axis of the welds

Effective Area of Welds

The effective area of groove welds is equal to the product of the width of the part joined and the effective throat thickness. The effective throat thickness of a full-penetration groove weld is taken as the thickness of the thinner part joined. The effective throat thickness of a partial-penetration groove weld is taken as the depth of the chamfer for J, U, bevel, or V (with bevel $\geq 60°$) joints and it is taken as the depth of the chamfer minus 1/8 in. for bevel or V joints if the bevel is between 45° and 60°. For flare bevel groove welds the effective throat thickness is taken as $5R/16$ and for flare V-groove the effective throat thickness is taken as $R/2$ (or $3R/8$ for GMAW process when $R \geq 1$ in.). R is the radius of the bar or bend.

The effective area of fillet welds is equal to the product of length of the fillets including returns and the effective throat thickness. The effective throat thickness of a fillet weld is the shortest distance from the root of the joint to the face of the diagrammatic weld as shown in Figure 3.21. Thus, for
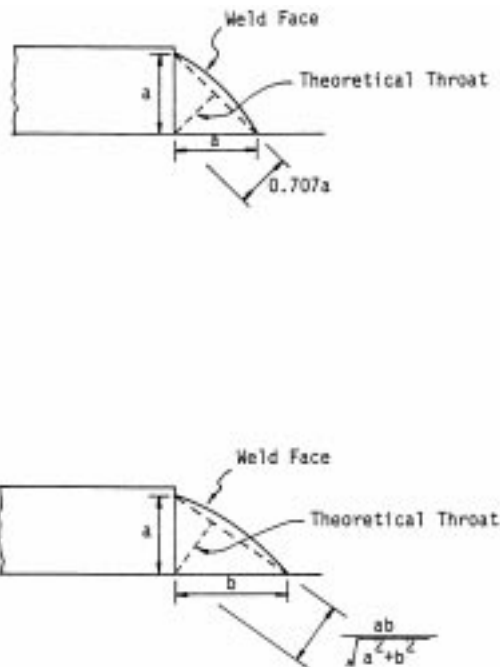
FIGURE 3.21: Effective throat of fillet welds.

an equal leg fillet weld, the effective throat is given by 0.707 times the leg dimension. For fillet weld made by the submerged arc welding process (SAW), the effective throat thickness is taken as the leg size (for 3/8-in. and smaller fillet welds) or as the theoretical throat plus 0.11-in. (for fillet weld over 3/8-in.). A larger value for the effective throat thickness is permitted for welds made by the SAW process to account for the inherently superior quality of such welds.

The effective area of plug and slot welds is taken as the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

Size and Length Limitations of Welds

To ensure effectiveness, certain size and length limitations are imposed for welds. For partial-penetration groove welds, minimum values for the effective throat thickness are given in Table 3.19.

TABLE 3.19 Minimum Effective Throat Thickness of Partial-Penetration Groove Welds

Thickness of the thicker part joined, t (in.)	Minimum effective throat thickness (in.)
$t \leq 1/4$	1/8
$1/4 < t \leq 1/2$	3/16
$1/2 < t \leq 3/4$	1/4
$3/4 < t \leq 1\text{-}1/2$	5/16
$1\text{-}1/2 < t \leq 2\text{-}1/4$	3/8
$2\text{-}1/4 < t \leq 6$	1/2
> 6	5/8

Note: 1 in. = 25.4 mm.

For fillet welds, the following size and length limitations apply:

Minimum Size of Leg—The minimum leg size is given in Table 3.20.

TABLE 3.20 Minimum Leg Size of Fillet Welds

Thickness of thicker part joined, t (in.)	Minimum leg size (in.)
$\leq 1/4$	1/8
$1/4 < t \leq 1/2$	3/16
$1/2 < t \leq 3/4$	1/4
$> 3/4$	5/16

Note: 1 in. = 25.4 mm.

Maximum Size of Leg—Along the edge of a connected part less than 1/4 thick, the maximum leg size is equal to the thickness of the connected part. For thicker parts, the maximum leg size is t minus 1/16 in. where t is the thickness of the part.

Minimum effective length of weld—The minimum effective length of a fillet weld is four times its nominal size. If a shorter length is used, the leg size of the weld shall be taken as 1/4 its effective length for purpose of stress computation. The length of fillet welds used for flat bar tension members shall not be less than the width of the bar if the welds are provided in the longitudinal direction only. The transverse distance between longitudinal welds should not exceed 8 in. unless the effect of shear lag is accounted for by the use of an effective net area.

Maximum effective length of weld—The maximum effective length of a fillet weld loaded by forces parallel to the weld shall not exceed 70 times the size of the fillet weld leg.

End returns—End returns must be continued around the corner and must have a length of at least two times the size of the weld leg.

Welded Connections for Tension Members

Figure 3.22 shows a tension angle member connected to a gusset plate by fillet welds. The applied tensile force P is assumed to act along the center of gravity of the angle. To avoid eccentricity, the lengths of the two fillet welds must be proportioned so that their resultant will also act along the
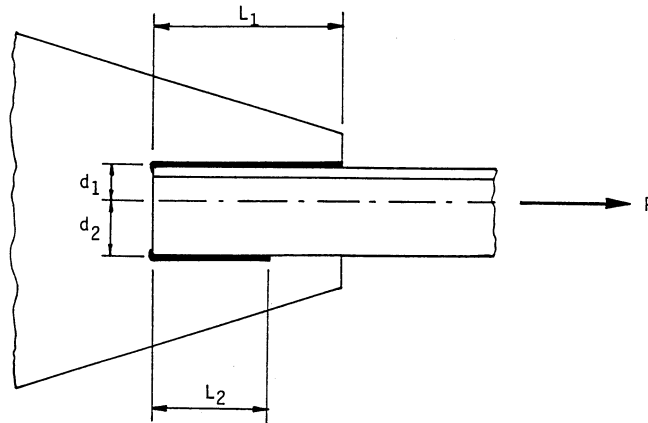
FIGURE 3.22: An eccentrically loaded welded tension connection.

center of gravity of the angle. For example, if LRFD is used, the following equilibrium equations can be written:

Summing force along the axis of the angle

$$(\phi F_M)t_{eff}L_1 + (\phi F_m)t_{eff}L_2 = P_u \tag{3.104}$$

Summing moment about the center of gravity of the angle

$$(\phi F_M)t_{eff}L_1d_1 = (\phi F_M)t_{eff}L_2d_2 \tag{3.105}$$

where P_u is the factored axial force, ϕF_M is the design strength of the welds as given in Table 3.18, t_{eff} is the effective throat thickness, L_1, L_2 are the lengths of the welds, and d_1, d_2 are the transverse distances from the center of gravity of the angle to the welds. The two equations can be used to solve for L_1 and L_2. If end returns are used, the added strength of the end returns should also be included in the calculations.

Welded Bracket Type Connections

A typical welded bracket connection is shown in Figure 3.23. Because the load is eccentric with respect to the center of gravity of the weld group, the connection is subjected to both moment and shear. The welds must be designed to resist the combined effect of direct shear for the applied load and any additional shear from the induced moment. The design of the welded bracket connection is facilitated by the use of design tables in the AISC-ASD and AISC-LRFD Manuals. In both ASD and LRFD, the load capacity for the connection is given by

$$P = CC_1Dl \tag{3.106}$$

where

P = allowable load (in ASD), or factored load, P_u (in LRFD), kips
l = length of the vertical weld, in.
D = number of sixteenths of an inch in fillet weld size
C_1 = coefficients for electrode used (see table below)
C = coefficients tabulated in the AISC-ASD and AISC-LRFD Manuals. In the tables, values of C for a variety of weld geometries and dimensions are given
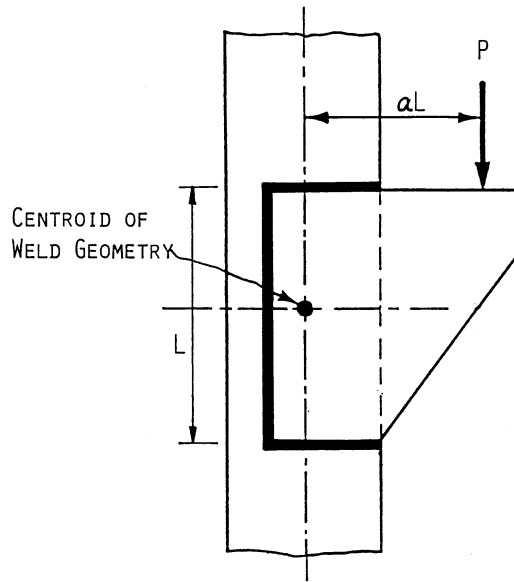
FIGURE 3.23: An eccentrically loaded welded bracket connection.

Electrode		E60	E70	E80	E90	E100	E110
ASD	F_v (ksi)	18	21	24	27	30	33
	C_1	0.857	1.0	1.14	1.29	1.43	1.57
LRFD	F_{EXX} (ksi)	60	70	80	90	100	110
	C_1	0.857	1.0	1.03	1.16	1.21	1.34

Welded Connections with Welds Subjected to Combined Shear and Flexure

Figure 3.24 shows a welded framed connection and a welded seated connection. The welds for these connections are subjected to combined shear and flexure. For purpose of design, it is common practice to assume that the shear force per unit length, R_S, acting on the welds is a constant and is given by

$$R_S = \frac{P}{2l} \tag{3.107}$$

where P is the allowable load (in ASD), or factored load, P_u (in LRFD), and l is the length of the vertical weld.

In addition to shear, the welds are subjected to flexure as a result of load eccentricity. There is no general agreement on how the flexure stress should be distributed on the welds. One approach is to assume that the stress distribution is linear with half the weld subjected to tensile flexure stress and half is subjected to compressive flexure stress. Based on this stress distribution and ignoring the returns, the flexure tension force per unit length of weld, R_F, acting at the top of the weld can be written as

$$R_F = \frac{Mc}{I} = \frac{P_e(l/2)}{2l^3/12} = \frac{3P_e}{l^2} \tag{3.108}$$

where e is the load eccentricity.

The resultant force per unit length acting on the weld, R, is then

$$R = \sqrt{R_S^2 + R_F^2} \tag{3.109}$$
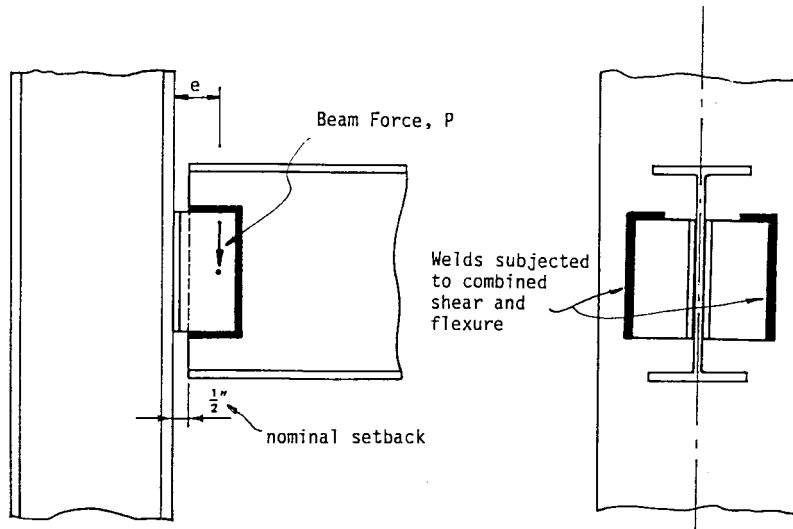
WELDED FRAMED CONNECTION
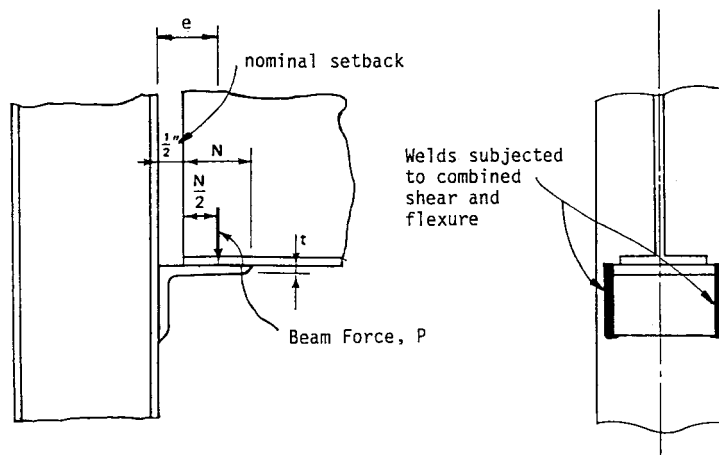
WELDED SEATED CONNECTION

FIGURE 3.24: Welds subjected to combined shear and flexure.

For a satisfactory design, the value R/t_{eff} where t_{eff} is the effective throat thickness of the weld should not exceed the allowable values or design strengths given in Table 3.18.

Welded Shear Connections

Figure 3.25 shows three commonly used welded shear connections: a framed beam connection, a seated beam connection, and a stiffened seated beam connection. These connections can be designed by using the information presented in the earlier sections on welds subjected to eccentric shear and welds subjected to combined tension and flexure. For example, the welds that connect the angles to the beam web in the framed beam connection can be considered as eccentrically loaded welds and so Equation 3.106 can be used for their design. The welds that connect the angles to the column flange can be considered as welds subjected to combined tension and flexure and so Equation 3.109 can be used for their design. Like bolted shear connections, welded shear connections are expected to exhibit appreciable ductility and so the use of angles with thickness in excess of 5/8 in. should be avoided. To prevent shear rupture failure, the shear rupture strength of the critically loaded connected parts should be checked.

To facilitate the design of these connections, the AISC-ASD and AISC-LRFD Manuals provide design tables by which the weld capacities and shear rupture strengths for different connection dimensions can be checked readily.

Welded Moment-Resisting Connections

Welded moment-resisting connections (Figure 3.26), like bolted moment-resisting connections, must be designed to carry both moment and shear. To simplify the design procedure, it is customary to assume that the moment, to be represented by a couple F_f as shown in Figure 3.19, is to be carried by the beam flanges and that the shear is to be carried by the beam web. The connected parts (e.g., the moment plates, welds, etc.) are then designed to resist the forces F_f and shear. Depending on the geometry of the welded connection, this may include checking: (a) the yield and/or fracture strength of the moment plate, (b) the shear and/or tensile capacity of the welds, and (c) the shear rupture strength of the shear plate.

If the column to which the connection is attached is weak, the designer should consider the use of column stiffeners to prevent failure of the column flange and web due to bending, yielding, crippling, or buckling (see section on Design of Moment-Resisting Connections).

Examples on the design of a variety of welded shear and moment-resisting connections can be found in the AISC Manual on Connections [20] and the AISC-LRFD Manual [22].

3.11.3 Shop Welded-Field Bolted Connections

A large percentage of connections used for construction are shop welded and field bolted types. These connections are usually more cost effective than fully welded connections and their strength and ductility characteristics often rival those of fully welded connections. Figure 3.27 shows some of these connections. The design of shop welded–field bolted connections is also covered in the AISC Manual on Connections and the AISC-LRFD Manual. In general, the following should be checked: (a) Shear/tensile capacities of the bolts and/or welds, (b) bearing strength of the connected parts, (c) yield and/or fracture strength of the moment plate, and (d) shear rupture strength of the shear plate. Also, as for any other types of moment connections, column stiffeners shall be provided if any one of the following criteria is violated: column flange bending, local web yielding, crippling, and compression buckling of the column web.

(a) Welded Framed Beam Connection

(b) Welded Seated Beam Connection

Stiffeners
finished to
bear

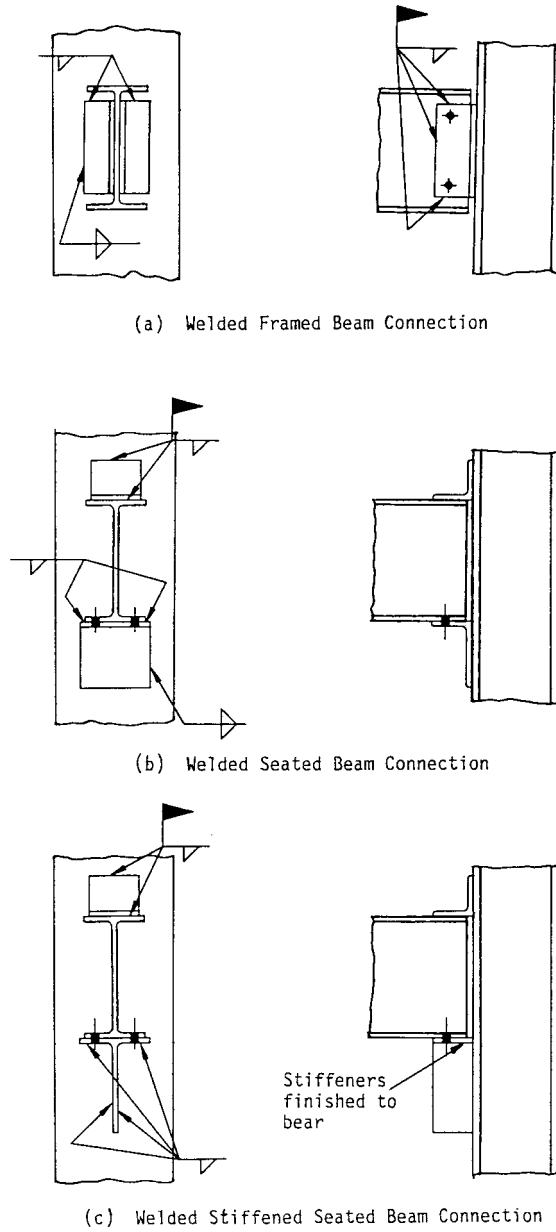(c) Welded Stiffened Seated Beam Connection

FIGURE 3.25: Welded shear connections. (a) Framed beam connection, (b) seated beam connection, (c) stiffened beam connection.
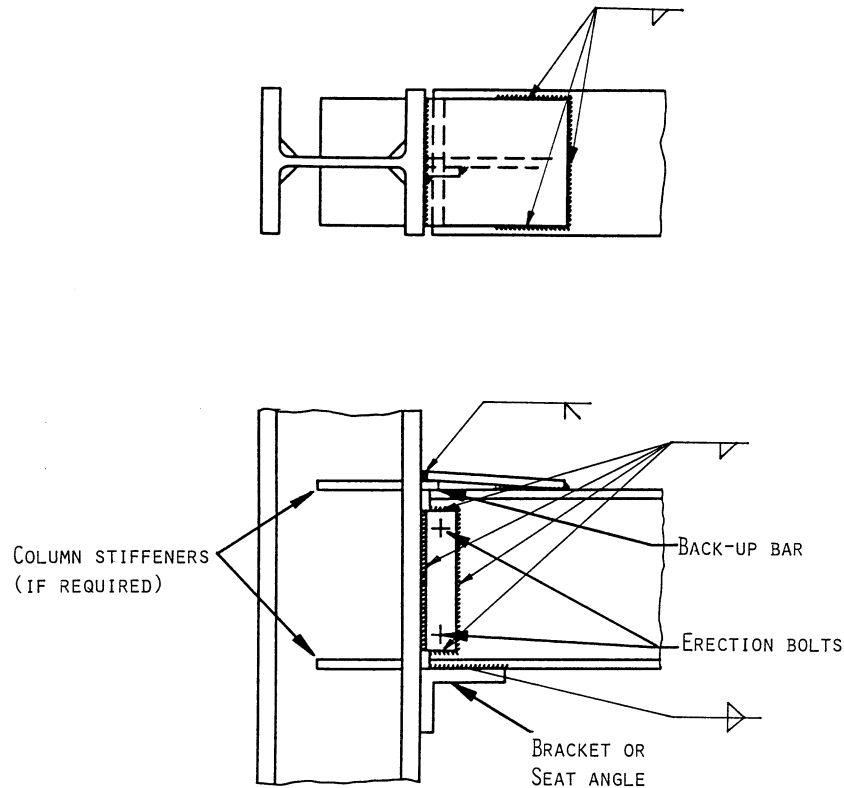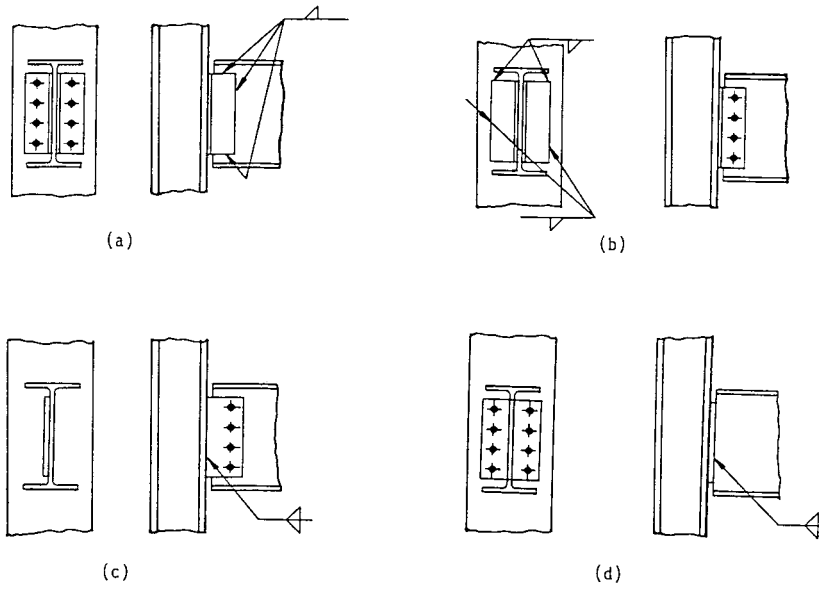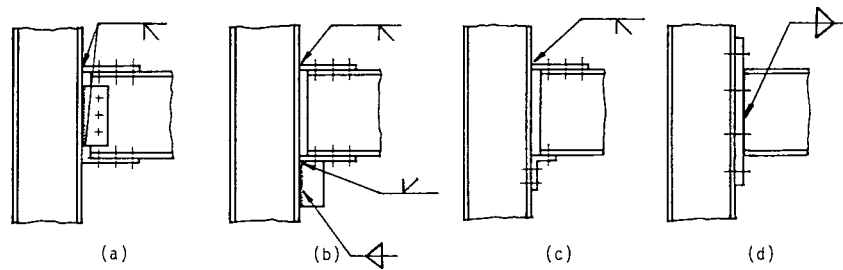
FIGURE 3.26: Welded moment connections.

3.11.4 Beam and Column Splices

Beam and column splices (Figure 3.28) are used to connect beam or column sections of different sizes. They are also used to connect beams or columns of the same size if the design calls for an extraordinarily long span. Splices should be designed for both moment and shear unless it is the intention of the designer to utilize the splices as internal hinges. If splices are used for internal hinges, provisions must be made to ensure that the connections possess adequate ductility to allow for large hinge rotation.

Splice plates are designed according to their intended functions. Moment splices should be designed to resist the flange force $F_f = M/(d - t_f)$ (Figure 3.19) at the splice location. In particular, the following limit states need to be checked: yielding of gross area of the plate, fracture of net area of the plate (for bolted splices), bearing strengths of connected parts (for bolted splices), shear capacity of bolts (for bolted splices), and weld capacity (for welded splices). Shear splices should be designed to resist the shear forces acting at the locations of the splices. The limit states that need to be checked include: shear rupture of the splice plates, shear capacity of bolts under an eccentric load (for bolted splices), bearing capacity of the connected parts (for bolted splices), shear capacity of bolts (for bolted splices), and weld capacity under an eccentric load (for welded splices). Design examples of beam and column splices can be found in the AISC Manual of Connections [20] and the AISC-LRFD Manuals [22].

SHOP WELDED-FIELD BOLTED SHEAR CONNECTIONS

SHOP WELDED-FIELD BOLTED MOMENT CONNECTIONS

FIGURE 3.27: Shop-welded field-bolted connections.
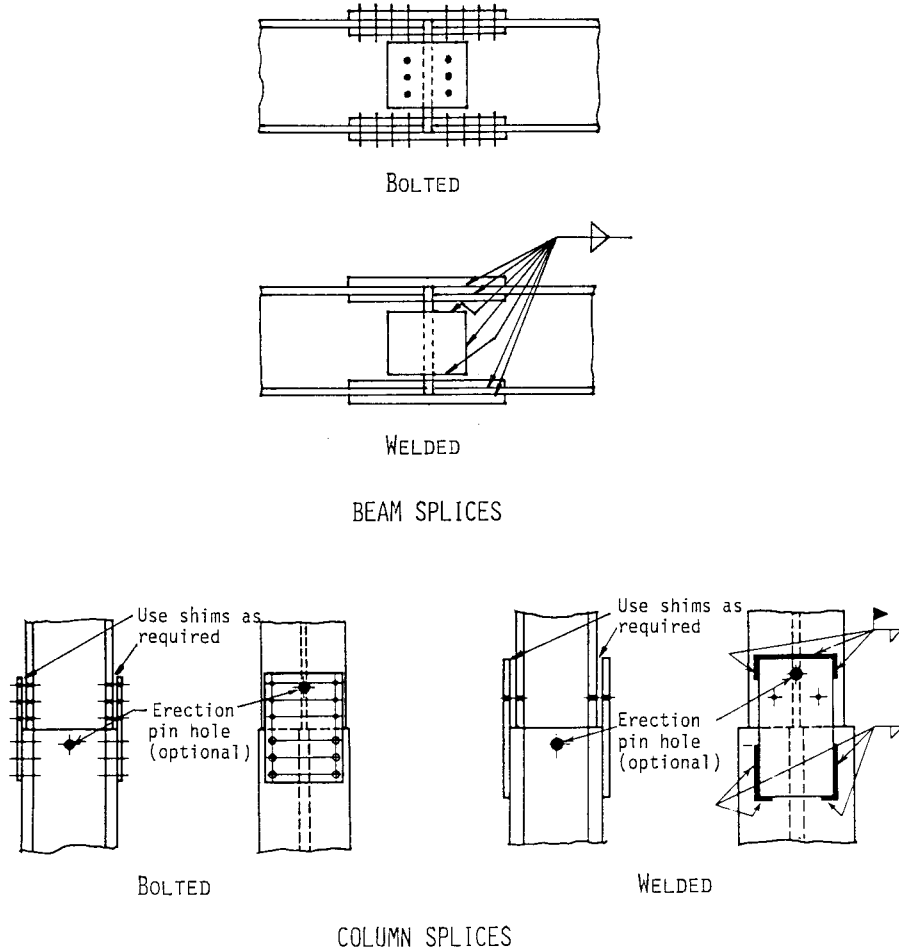
FIGURE 3.28: Bolted and welded beam and column splices.

3.12 Column Base Plates and Beam Bearing Plates (LRFD Approach)

3.12.1 Column Base Plates

Column base plates are steel plates placed at the bottom of columns whose function is to transmit column loads to the concrete pedestal. The design of column base plates involves two major steps: (1) determining the size $N \times B$ of the plate, and (2) determining the thickness t_p of the plate. Generally, the size of the plate is determined based on the limit state of bearing on concrete and the thickness of the plate is determined based on the limit state of plastic bending of critical sections in the plate. Depending on the types of forces (axial force, bending moment, shear force) the plate will be subjected to, the design procedures differ slightly. In all cases, a layer of grout should be placed between the base plate and its support for the purpose of leveling and anchor bolts should be provided to stabilize the column during erection or to prevent uplift for cases involving large bending moment.

Axially Loaded Base Plates

Base plates supporting concentrically loaded columns in frames in which the column bases are assumed pinned are designed with the assumption that the column factored load P_u is distributed uniformly to the area of concrete under the base plate. The size of the base plate is determined from the limit state of bearing on concrete. The design bearing strength of concrete is given by the equation

$$\phi_c P_p = 0.60 \left[0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}} \right] \tag{3.110}$$

where
f_c' = compressive strength of concrete
A_1 = area of base plate
A_2 = area of concrete pedestal that is geometrically similar to and concentric with the loaded area, $A_1 \leq A_2 \leq 4A_1$

From Equation 3.110, it can be seen that the bearing capacity increases when the concrete area is greater than the plate area. This accounts for the beneficial effect of confinement. The upper limit of the bearing strength is obtained when $A_2 = 4A_1$. Presumably, the concrete area in excess of $4A_1$ is not effective in resisting the load transferred through the base plate.

Setting the column factored load, P_u, equal to the bearing capacity of the concrete pedestal, $\phi_c P_p$, and solving for A_1 from Equation 3.110, we have

$$A_1 = \frac{1}{A_2} \left[\frac{P_u}{0.6(0.85 f_c')} \right]^2 \tag{3.111}$$

The length, N, and width, B, of the plate should be established so that $N \times B > A_1$. For an efficient design, the length can be determined from the equation

$$N \approx \sqrt{A_1} + 0.50(0.95d - 0.80b_f) \tag{3.112}$$

where $0.95d$ and $0.80b_f$ define the so-called effective load bearing area shown cross-hatched in Figure 3.29a. Once N is obtained, B can be solved from the equation

$$B = \frac{A_1}{N} \tag{3.113}$$

Both N and B should be rounded up to the nearest full inches.

The required plate thickness, $t_{reg'd}$, is to be determined from the limit state of yield line formation along the most severely stressed sections. A yield line develops when the cross-section moment capacity is equal to its plastic moment capacity. Depending on the size of the column relative to the plate and the magnitude of the factored axial load, yield lines can form in various patterns on the plate. Figure 3.29 shows three models of plate failure in axially loaded plates. If the plate is large compared to the column, yield lines are assumed to form around the perimeter of the effective load bearing area (the cross-hatched area) as shown in Figure 3.29a. If the plate is small and the column factored load is light, yield lines are assumed to form around the inner perimeter of the I-shaped area as shown in Figure 3.29b. If the plate is small and the column factored load is heavy, yield lines are assumed to form around the inner edge of the column flanges and both sides of the column web as shown in Figure 3.29c. The following equation can be used to calculate the required plate thickness
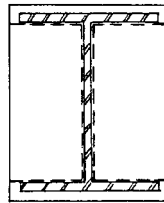
$$t_{req'd} = l \sqrt{\frac{2 P_u}{0.90 F_y B N}} \tag{3.114}$$
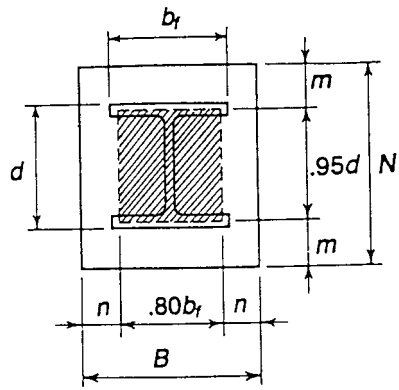
(a) Plate with Large m,n
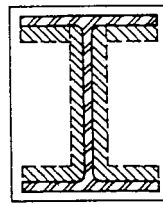
(b) Lightly Loaded Plate with Small m, n

(c) Heavily Loaded Plate with Small m, n

FIGURE 3.29: Failure models for centrally loaded column base plates.

where l is the larger of m, n, and $\lambda n'$ given by

$$
\begin{aligned}
m &= \frac{(N - 0.95d)}{2} \\
n &= \frac{(B - 0.80b_f)}{2} \\
n' &= \frac{\sqrt{db_f}}{4}
\end{aligned}
$$

and

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1-X}} \leq 1$$

in which

$$X = \left(\frac{4db_f}{(d+b_f)^2}\right)\frac{P_u}{\phi_c P_p}$$

Base Plates for Tubular and Pipe Columns

The design concept for base plates discussed above for I-shaped sections can be applied to the design of base plates for rectangular tubes and circular pipes. The critical section used to determine the plate thickness should be based on 0.95 times the outside column dimension for rectangular tubes and 0.80 times the outside dimension for circular pipes [11].

Base Plates with Moments

For columns in frames designed to carry moments at the base, base plates must be designed to support both axial forces and bending moments. If the moment is small compared to the axial force, the base plate can be designed without consideration of the tensile force which may develop in the anchor bolts. However, if the moment is large, this effect should be considered. To quantify the relative magnitude of this moment, an eccentricity $e = M_u/P_u$ is used. The general procedures for the design of base plates for different values of e will be given in the following [11].

Small eccentricity, $e \leq N/6$

If e is small, the bearing stress is assumed to distribute linearly over the entire area of the base plate (Figure 3.30). The maximum bearing stress is given by

$$f_{\max} = \frac{P_u}{BN} + \frac{M_u c}{I} \tag{3.115}$$

where $c = N/2$ and $I = BN^3/12$.
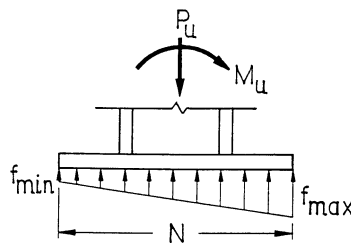
FIGURE 3.30: Eccentrically loaded column base plate (small load eccentricity).

The size of the plate is to be determined by a trial and error process. The size of the base plate should be such that the bearing stress calculated using Equation 3.115 does not exceed $\phi_c P_p/A_1$,

given by

$$0.60 \left[0.85 f_c' \sqrt{\frac{A_2}{A_1}} \right] \le 0.60[1.7 f_c'] \tag{3.116}$$

The thickness of the plate is to be determined from

$$t_p = \sqrt{\frac{4M_{plu}}{0.90 F_y}} \tag{3.117}$$

where M_{plu} is the moment per unit width of critical section in the plate. M_{plu} is to be determined by assuming that the portion of the plate projecting beyond the critical section acts as an inverted cantilever loaded by the bearing pressure. The moment calculated at the critical section divided by the length of the critical section (i.e., B) gives M_{plu}.

Moderate eccentricity, $N/6 < e \le N/2$
    For plates subjected to moderate moments, only portions of the plate will be subjected to bearing stress (Figure 3.31). Ignoring the tensile force in the anchor bolt in the region of the plate where no
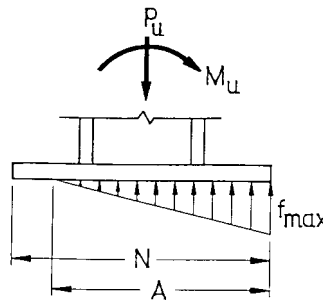
FIGURE 3.31: Eccentrically loaded column base plate (moderate load eccentricity).

bearing occurs and denoting A as the length of the plate in bearing, the maximum bearing stress can be calculated from force equilibrium consideration as

$$f_{\max} = \frac{2P_u}{AB} \tag{3.118}$$

where $A = 3(N/2 - e)$ is determined from moment equilibrium. The plate should be portioned such that $f_{\max}$ does not exceed the value calculated using Equation 3.116. t_p is to be determined from Equation 3.117.

Large eccentricity, $e > N/2$
    For plates subjected to large bending moments so that $e > N/2$, one needs to take into consideration the tensile force developing in the anchor bolts (Figure 3.32). Denoting T as the resultant force in the anchor bolts, force equilibrium requires that

$$T + P_u = \frac{f_{\max} AB}{2} \tag{3.119}$$
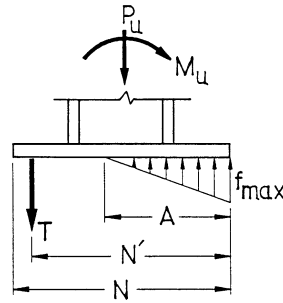
FIGURE 3.32: Eccentrically loaded column base plate (large load eccentricity).

and moment equilibrium requires that

$$P_u \left(N' - \frac{N}{2} \right) + M = \frac{f_{\max} A B}{2} \left(N' - \frac{A}{3} \right) \tag{3.120}$$

The above equations can be used to solve for A and T. The size of the plate is to be determined using a trial-and-error process. The size should be chosen such that $f_{\max}$ does not exceed the value calculated using Equation 3.116, A should be smaller than N' and T should not exceed the tensile capacity of the bolts.

Once the size of the plate is determined, the plate thickness t_p is to be calculated using Equation 3.117. Note that there are two critical sections on the plate, one on the compression side of the plate and the other on the tension side of the plate. Two values of M_{plu} are to be calculated and the larger value should be used to calculate t_p.

Base Plates with Shear

Under normal circumstances, the factored column base shear is adequately resisted by the frictional force developed between the plate and its support. Additional shear capacity is also provided by the anchor bolts. For cases in which exceptionally high shear force is expected, such as in a bracing connection or in which uplift occurs which reduces the frictional resistance, the use of shear lugs may be necessary. Shear lugs can be designed based on the limit states of bearing on concrete and bending of the lugs. The size of the lug should be proportioned such that the bearing stress on concrete does not exceed $0.60(0.85 f'_c)$. The thickness of the lug can be determined from Equation 3.117. M_{plu} is the moment per unit width at the critical section of the lug. The critical section is taken to be at the junction of the lug and the plate (Figure 3.33).

3.12.2   Anchor Bolts

Anchor bolts are provided to stabilize the column during erection and to prevent uplift for cases involving large moments. Anchor bolts can be cast-in-place bolts or drilled-in bolts. The latter are placed after the concrete is set and are not too often used. Their design is governed by the manufacturer's specifications. Cast-in-place bolts are hooked bars, bolts, or threaded rods with nuts (Figure 3.34) placed before the concrete is set. Of the three types of cast-in-place anchors shown in the figure, the hooked bars are recommended for use only in axially loaded base plates. They are not normally relied upon to carry significant tensile force. Bolts and threaded rods with nuts can be used
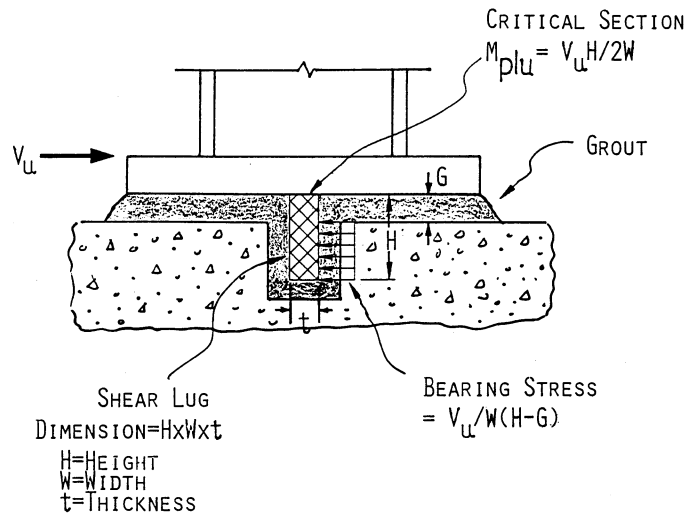
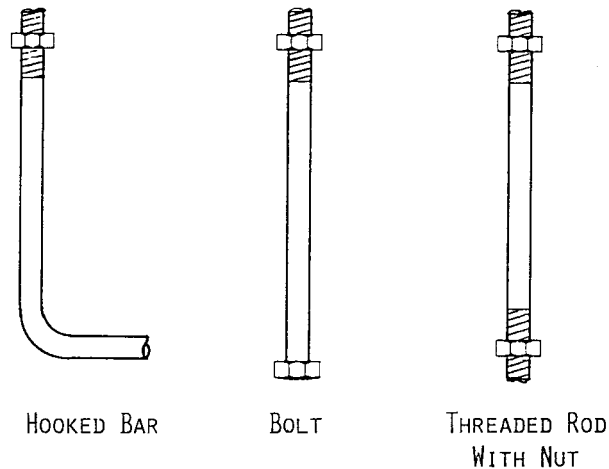FIGURE 3.33: Column base plate subjected to shear.

FIGURE 3.34: Base plate anchors.

for both axially loaded base plates or base plates with moments. Threaded rods with nuts are used when the length and size required for the specific design exceed those of standard size bolts. Failure of bolts or threaded rods with nuts occur when their tensile capacities are reached. Failure is also considered to occur when a cone of concrete is pulled out from the pedestal. This cone pull-out type of failure is depicted schematically in Figure 3.35. The failure cone is assumed to radiate out from the bolt head or nut at an angle of 45° with tensile failure occurring along the surface of the cone at an average stress of $4\sqrt{f'_c}$ where f'_c is the compressive strength of concrete in psi. The load that will cause this cone pull-out failure is given by the product of this average stress and the projected area the cone A_p [23, 24]. The design of anchor bolts is thus governed by the limit states of tensile fracture of the anchors and cone pull-out.
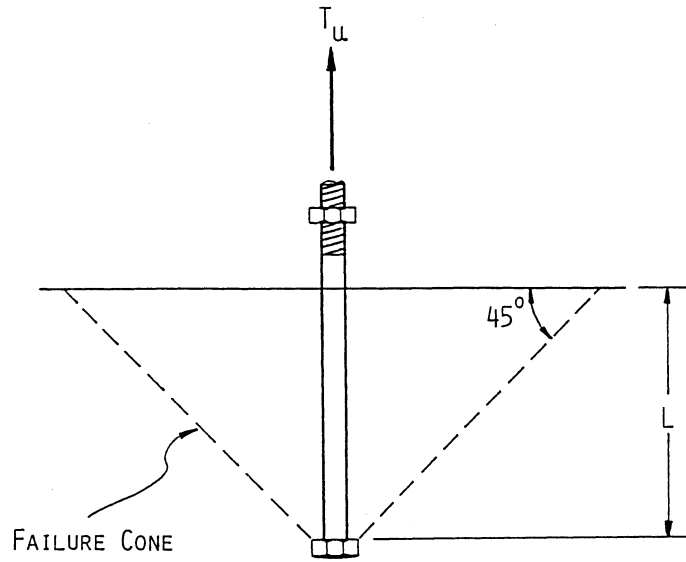
FIGURE 3.35: Cone pullout failure.

Limit State of Tensile Fracture

The area of the anchor should be such that

$$A_g \geq \frac{T_u}{\phi_t 0.75 F_u} \tag{3.121}$$

where A_g is the required gross area of the anchor, F_u is the minimum specified tensile strength, and ϕ_t is the resistance factor for tensile fracture which is equal to 0.75.

Limit State of Cone Pull-Out

From Figure 3.35, it is clear that the size of the cone is a function of the length of the anchor. Provided that there is sufficient edge distance and spacing between adjacent anchors, the amount of tensile force required to cause cone pull-out failure increases with the embedded length of the anchor. This concept can be used to determine the required embedded length of the anchor. Assuming that the failure cone does not intersect with another failure cone nor the edge of the pedestal, the required embedded length can be calculated from the equation

$$L \geq \sqrt{\frac{A_p}{\pi}} = \sqrt{\frac{(T_u/\phi_t 4\sqrt{f_c'})}{\pi}} \tag{3.122}$$

where A_p is the projected area of the failure cone, T_u is the required bolt force in pounds, f_c' is the compressive strength of concrete in psi and ϕ_t is the resistance factor assumed to be equal to 0.75. If failure cones from adjacent anchors overlap one another or intersect with the pedestal edge, the projected area A_p must be adjusted according (see, for example [23, 24]).

The length calculated using the above equation should not be less than the recommended values given by [29]. These values are reproduced in the following table. Also shown in the table are the recommended minimum edge distances for the anchors.

Bolt type (material)	Minimum embedded length	Minimum edge distance
A307 (A36)	$12d$	$5d > 4$ in.
A325 (A449)	$17d$	$7d > 4$ in.

d = nominal diameter of the anchor

3.12.3 Beam Bearing Plates

Beam bearing plates are provided between main girders and concrete pedestals to distribute the girder reactions to the concrete supports (Figure 3.36). Beam bearing plates may also be provided between cross beams and girders if the cross beams are designed to sit on the girders.
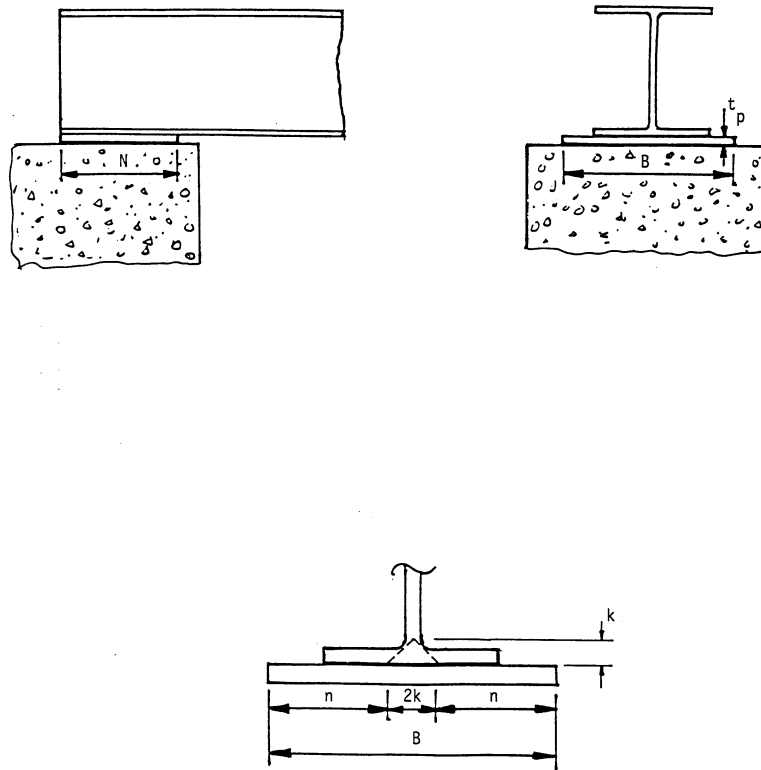

FIGURE 3.36: Beam bearing plate.

Beam bearing plates are designed based on the limit states of web yielding, web crippling, bearing on concrete, and plastic bending of the plate. The dimension of the plate along the beam axis, i.e., N, is determined from the web yielding or web crippling criterion (see section on Concentrated Load Criteria), whichever is more critical. The dimension B of the plate is determined from Equation 3.113 with A_1 calculated using Equation 3.111. P_u in Equation 3.111 is to be replaced by R_u, the factored reaction at the girder support.

Once the size $B \times N$ is determined, the plate thickness t_p can be calculated using the equation

$$t_p = \sqrt{\frac{2R_u n^2}{0.90 F_y B N}} \qquad (3.123)$$

where R_u is the factored girder reaction, F_y is the yield stress of the plate and $n = (B - 2k)/2$ in which k is the distance from the web toe of the fillet to the outer surface of the flange. The above equation was developed based on the assumption that the critical sections for plastic bending in the plate occur at a distance k from the centerline of the web.

3.13 Composite Members (LRFD Approach)

Composite members are structural members made from two or more materials. The majority of composite sections used for building constructions are made from steel and concrete. Steel provides strength and concrete provides rigidity. The combination of the two materials often results in efficient load-carrying members. Composite members may be concrete-encased or concrete-filled. For concrete-encased members (Figure 3.37a), concrete is casted around steel shapes. In addition to enhancing strength and providing rigidity to the steel shapes, the concrete acts as a fire-proofing material to the steel shapes. It also serves as a corrosion barrier shielding the steel from corroding under adverse environmental conditions. For concrete-filled members (Figure 3.37b), structural steel tubes are filled with concrete. In both concrete-encased and concrete-filled sections, the rigidity of the concrete often eliminates the problem of local buckling experienced by some slender elements of the steel sections.

Some disadvantages associated with composite sections are that concrete creeps and shrinks. Furthermore, uncertainties with regard to the mechanical bond developed between the steel shape and the concrete often complicate the design of beam-column joints.
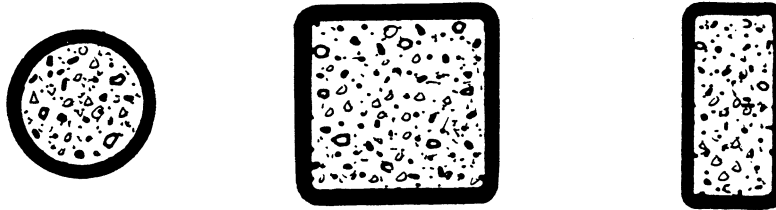
3.13.1 Composite Columns

According to the LRFD Specification [18], a compression member is regarded as a composite column if (1) the cross-sectional area of the steel shape is at least 4% of the total composite area. If this condition is not satisfied, the member should be designed as a reinforced concrete column. (2) Longitudinal reinforcements and lateral ties are provided for concrete-encased members. The cross-sectional area of the reinforcing bars shall be 0.007 in.^{2} per inch of bar spacing. To avoid spalling, lateral ties shall be placed at a spacing not greater than 2/3 the least dimension of the composite cross-section. For fire and corrosion resistance, a minimum clear cover of 1.5 in. shall be provided. (3) The compressive strength of concrete f_c' used for the composite section falls within the range 3 to 8 ksi for normal weight concrete and not less than 4 ksi for light weight concrete. These limits are set because they represent the range of test data available for the development of the design equations. (4) The specified minimum yield stress for the steel shapes and reinforcing bars used in calculating the strength of the composite column does not exceed 55 ksi. This limit is set because this stress corresponds to a strain below which the concrete remains unspalled and stable. (5) The minimum wall thickness of the steel shapes for concrete filled members is equal to $b\sqrt{(F_y/3E)}$ for rectangular sections of width b and $D\sqrt{(F_y/8E)}$ for circular sections of outside diameter D.

Design Compressive Strength

The design compressive strength, $\phi_c P_n$, shall exceed the factored compressive force, P_u. The design compressive strength is given as follows:
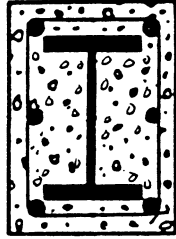
(a) CONCRETE ENCASED COMPOSITE SECTION

(b) CONCRETE FILLED COMPOSITE SECTIONS

FIGURE 3.37: Composite columns.

For $\lambda_c \leq 1.5$

$$\phi_c P_n = \begin{cases} 0.85 \left[\left(0.658^{\lambda_c^2} \right) A_s F_{my} \right], & \text{if } \lambda_c \leq 1.5 \\ 0.85 \left[\left(\frac{0.877}{\lambda_c^2} \right) A_s F_{my} \right], & \text{if } \lambda_c > 1.5 \end{cases} \qquad (3.124)$$

where

$$\lambda_c = \frac{KL}{r_m \pi} \sqrt{\frac{F_{my}}{E_m}} \qquad (3.125)$$

$$F_{my} = F_y + c_1 F_{yr} \left(\frac{A_r}{A_s} \right) + c_2 f_c' \left(\frac{A_c}{A_s} \right) \qquad (3.126)$$

$$E_m = E + c_3 E_c \left(\frac{A_c}{A_s} \right) \qquad (3.127)$$

A_c = area of concrete, in.^{2}
A_r = area of longitudinal reinforcing bars, in.^{2}
A_s = area of steel shape, in.^{2}
E = modulus of elasticity of steel, ksi
E_c = modulus of elasticity of concrete, ksi
F_y = specified minimum yield stress of steel shape, ksi
F_{yr} = specified minimum yield stress of longitudinal reinforcing bars, ksi

f_c' = specified compressive strength of concrete, ksi
c_1, c_2, c_3 = coefficients given in table below

Type of composite section	c_1	c_2	c_3
Concrete encased shapes	0.7	0.6	0.2
Concrete-filled pipes and tubings	1.0	0.85	0.4

In addition to satisfying the condition $\phi_c P_n \geq P_u$, the bearing condition for concrete must also be satisfied. Denoting $\phi_c P_{nc} (= \phi_c P_{n,\text{composite section}} - \phi_c P_{n,\text{steel shape alone}})$ as the portion of compressive strength resisted by the concrete and A_B as the loaded area (the condition), then if the supporting concrete area is larger than the loaded area, the bearing condition that needs to be satisfied is

$$\phi_c P_{nc} \leq 0.60[1.7 f_c' A_B] \tag{3.128}$$

3.13.2 Composite Beams

For steel beams fully encased in concrete, no additional anchorage for shear transfer is required if (1) at least 1.5 in. concrete cover is provided on top of the beam and at least 2 in. cover is provided over the sides and at the bottom of the beam, and (2) spalling of concrete is prevented by adequate mesh or other reinforcing steel. The design flexural strength $\phi_b M_n$ can be computed using either an elastic or plastic analysis.

If an elastic analysis is used, ϕ_b shall be taken as 0.90. A linear strain distribution is assumed for the cross-section with zero strain at the neutral axis and maximum strains at the extreme fibers. The stresses are then computed by multiplying the strains by E (for steel) or E_c (for concrete). Maximum stress in steel shall be limited to F_y, and maximum stress in concrete shall be limited to $0.85 f_c'$. Tensile strength of concrete shall be neglected. M_n is to be calculated by integrating the resulting stress block about the neutral axis.

If a plastic analysis is used, ϕ_c shall be taken as 0.90, and M_n shall be assumed to be equal to M_p, the plastic moment capacity of the steel section alone.

3.13.3 Composite Beam-Columns

Composite beam-columns shall be designed to satisfy the interaction equation of Equation 3.68 or Equation 3.69, whichever is applicable, with $\phi_c P_n$ calculated based on Equations 3.124 to 3.127, P_e calculated using the equation $P_e = A_s F_{my}/\lambda_c^2$, and $\phi_b M_n$ calculated using the following equation [14]:

$$\phi_b M_n = 0.90 \left[Z F_y + \frac{1}{3}(h_2 - 2c_r) A_r F_{yr} + \left(\frac{h_2}{2} - \frac{A_w F_y}{1.7 f_c' h_1} \right) A_w F_y \right] \tag{3.129}$$

where
Z = plastic section modulus of the steel section, in.[3]
c_r = average of the distance measured from the compression face to the longitudinal reinforcement in that face and the distance measured from the tension face to the longitudinal reinforcement in that face, in.
h_1 = width of the composite section perpendicular to the plane of bending, in.
h_2 = width of the composite section parallel to the plane of bending, in.
A_r = cross-sectional area of longitudinal reinforcing bars, in.[2]
A_w = web area of the encased steel shape ($= 0$ for concrete-filled tubes)

If $0 < (P_u/\phi_c P_n) \leq 0.3$, a linear interpolation of $\phi_b M_n$ calculated using the above equation assuming $P_u/\phi_c P_n = 0.3$ and that for beams with $P_u/\phi_c P_n = 0$ (see section on Composite Beams) should be used.

3.13.4   Composite Floor Slabs

Composite floor slabs (Figure 3.38) can be designed as shored or unshored. In shored construction,

COMPOSITE FLOOR SLAB WITH STUD SHEAR CONNECTORS

COMPOSITE FLOOR SLAB WITH CHANNEL SHEAR CONNECTORS

COMPOSITE FLOOR SLAB WITH SPIRAL SHEAR CONNECTORS
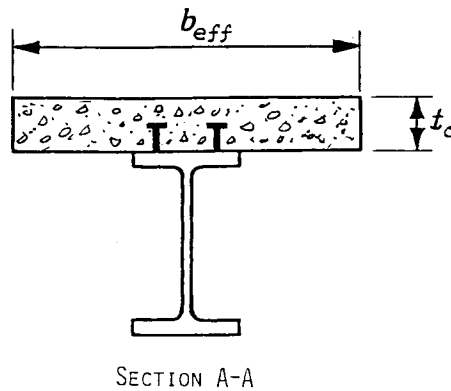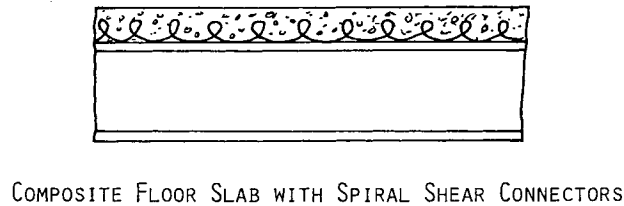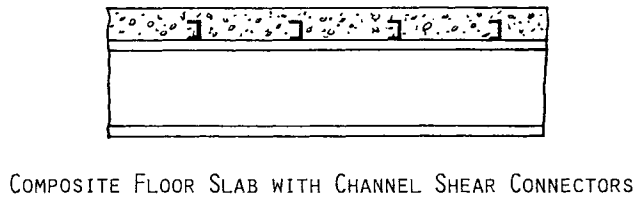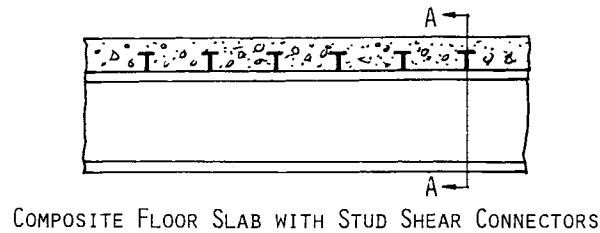
SECTION A-A

FIGURE 3.38: Composite floor slabs.

temporary shores are used during construction to support the dead and accidental live loads until the concrete cures. The supporting beams are designed on the basis of their ability to develop composite action to support all factored loads after the concrete cures. In unshored construction, temporary shores are not used. As a result, the steel beams alone must be designed to support the dead and accidental live loads before the concrete has attained 75% of its specified strength. After the concrete is cured, the composite section should have adequate strength to support all factored loads.

Composite action for the composite floor slabs shown in Figure 3.38 is developed as a result of the presence of shear connectors. If sufficient shear connectors are provided so that the maximum flexural strength of the composite section can be developed, the section is referred to as fully composite. Otherwise, the section is referred to as partially composite. The flexural strength of a partially composite section is governed by the shear strength of the shear connectors. The horizontal shear force V_h, which should be designed for at the interface of the steel beam and the concrete slab, is given by:

In regions of positive moment

$$V_h = \min(0.85 f_c' A_c, \, A_s F_y, \, \sum Q_n) \tag{3.130}$$

In regions of negative moment

$$V_h = \min(A_r F_{yr}, \, \sum Q_n) \tag{3.131}$$

where
f_c' = compressive strength of concrete, ksi
A_c = effective area of the concrete slab = $t_c b_{eff}$, in.^{2}
t_c = thickness of the concrete slab, in.
b_{eff} = effective width of the concrete slab, in.
　 = $\min(L/4, s)$, for an interior beam
　 = $\min(L/8+$ distance from beam centerline to edge of slab, $s/2+$ distance from beam centerline to edge of slab), for an exterior beam
L = beam span measured from center-to-center of supports, in.
s = spacing between centerline of adjacent beams, in.
A_s = cross-sectional area of the steel beam, in.^{2}
F_y = yield stress of the steel beam, ksi
A_r = area of reinforcing steel within the effective area of the concrete slab, in.^{2}
F_{yr} = yield stress of the reinforcing steel, ksi
ΣQ_n = sum of nominal shear strengths of the shear connectors, kips

The nominal shear strength of a shear connector (used without a formed steel deck) is given by:

For a stud shear connector

$$Q_n = 0.5 A_{sc}\sqrt{f_c' E_c} \le A_{sc} F_u \tag{3.132}$$

For a channel shear connector

$$Q_n = 0.3(t_f + 0.5 t_w) L_c \sqrt{f_c' E_c} \tag{3.133}$$

where
A_{sc} = cross-sectional area of the shear stud, in.^{2}
f_c' = compressive strength of concrete, ksi
E_c = modulus of elasticity of concrete, ksi

F_u = minimum specified tensile strength of the shear stud, ksi
t_f = flange thickness of the channel, in.
t_w = web thickness of the channel, in.
L_c = length of the channel, in.

If a formed steel deck is used, Q_n must be reduced by a reduction factor. The reduction factor depends on whether the deck ribs are perpendicular or parallel to the steel beam. Expressions for the reduction factor are given in the AISC-LRFD Specification [18].

For full composite action, the number of connectors required between the *maximum* moment point and the *zero* moment point of the beam is given by

$$N = \frac{V_h}{Q_n} \tag{3.134}$$

For partial composite action, the number of connectors required is governed by the condition $\phi_b M_n \geq M_u$, where $\phi_b M_n$ is governed by the shear strength of the connectors.

The placement and spacing of the shear connectors should comply with the following guidelines:

1. The shear connectors shall be uniformly spaced with the region of maximum moment and zero moment. However, the number of shear connectors placed between a concentrated load point and the nearest zero moment point must be sufficient to resist the factored moment M_u.
2. Except for connectors installed in the ribs of formed steel decks, shear connectors shall have at least 1 in. of lateral concrete cover.
3. Unless located over the web, diameter of shear studs must not exceed 2.5 times the thickness of the beam flange.
4. The longitudinal spacing of the studs should fall in the range 6 times the stud diameter to 8 times the slab thickness if a solid slab is used or 4 times the stud diameter to 8 times the slab thickness if a formed steel deck is used.

The design flexural strength $\phi_b M_n$ of the composite beam with shear connectors is determined as follows:

In regions of positive moments

For $h_c/t_w \leq 640/\sqrt{F_{yf}}$, $\phi_b = 0.85$, M_n = moment capacity determined using a plastic stress distribution assuming concrete crushes at a stress of $0.85 f_c'$ and steel yields at a stress of F_y. If a portion of the concrete slab is in tension, the strength contribution of that portion of concrete is ignored. The determination of M_n using this method is very similar to the technique used for computing the moment capacity of a reinforced concrete beam according to the ultimate strength method.

For $h_c/t_w > 640/\sqrt{F_{yf}}$, $\phi_b = 0.90$, M_n = moment capacity determined using superposition of elastic stress, considering the effect of shoring. The determination of M_n using this method is quite similar to the technique used for computing the moment capacity of a reinforced concrete beam according to the working stress method.

In regions of negative moments

$\phi_b M_n$ is to be determined for the steel section alone in accordance with the requirements discussed in the section on Flexural Members.

To facilitate design, numerical values of $\phi_b M_n$ for composite beams with shear studs in solid slabs are given in tabulated form by the AISC-LRFD Manual. Values of $\phi_b M_n$ for composite beams with formed steel decks are given in a publication by the Steel Deck Institute [19].

3.14 Plastic Design

Plastic analysis and design is permitted only for steels with yield stress not exceeding 65 ksi. The reason for this is that steels with high yield stress lack the ductility required for inelastic rotation at hinge locations. Without adequate inelastic rotation, moment redistribution (which is an important characteristic for plastic design) cannot take place.

In plastic design, the predominant limit state is the formation of plastic hinges. Failure occurs when sufficient plastic hinges have formed for a collapse mechanism to develop. To ensure that plastic hinges can form and can undergo large inelastic rotation, the following conditions must be satisfied:

1. Sections must be compact. That is, the width-thickness ratios of flanges in compression and webs must not exceed λ_p in Table 3.8.
2. For columns, the slenderness parameter λ_c (see section on Compression Members) shall not exceed $1.5K$ where K is the effective length factor, and P_u from gravity and horizontal loads shall not exceed $0.75A_g F_y$.
3. For beams, the lateral unbraced length L_b shall not exceed L_{pd} where

For doubly and singly symmetric I-shaped members loaded in the plane of the web

$$L_{pd} = \frac{3{,}600 + 2{,}200(M_1/M_2)}{F_y} r_y \tag{3.135}$$

and for solid rectangular bars and symmetric box beams

$$L_{pd} = \frac{5{,}000 + 3{,}000(M_1/M_2)}{F_y} r_y \geq \frac{3{,}000 r_y}{F_y} \tag{3.136}$$

In the above equations, M_1 is the smaller end moment within the unbraced length of the beam. $M_2 = M_p$ is the plastic moment ($= Z_x F_y$) of the cross-section. r_y is the radius of gyration about the minor axis, in inches, and F_y is the specified minimum yield stress, in ksi.

L_{pd} is not defined for beams bent about their minor axes nor for beams with circular and square cross-sections because these beams do not experience lateral torsional bucking when loaded.

3.14.1 Plastic Design of Columns and Beams

Provided that the above limitations are satisfied, the design of columns shall meet the condition $1.7F_a A \geq P_u$ where F_a is the allowable compressive stress given in Equation 3.16, A is the gross cross-sectional area, and P_u is the factored axial load.

The design of beams shall satisfy the conditions $M_p \geq M_u$ and $0.55F_y t_w d \geq V_u$ where M_u and V_u are the factored moment and shear, respectively. M_p is the plastic moment capacity F_y is the minimum specified yield stress, t_w is the beam web thickness, and d is the beam depth. For beams subjected to concentrated loads, all failure modes associated with concentrated loads (see section on Concentrated Load Criteria) should also be prevented.

Except at the location where the last hinge forms, a beam bending about its major axis must be braced to resist lateral and torsional displacements at plastic hinge locations. The distance between adjacent braced points should not exceed l_{cr} given by

$$l_{cr} = \begin{cases} \left(\dfrac{1375}{F_y} + 25\right) r_y, & \text{if } -0.5 < \dfrac{M}{M_p} < 1.0 \\[2ex] \left(\dfrac{1375}{F_y}\right) r_y, & \text{if } -1.0 < \dfrac{M}{M_p} \leq -0.5 \end{cases} \tag{3.137}$$

where

r_y =   radius of gyration about the weak axis
M =   smaller of the two end moments of the unbraced segment
M_p =   plastic moment capacity
M/M_p =   is taken as positive if the unbraced segment bends in reverse curvature, and it is taken as
          negative if the unbraced segment bends in single curvature

3.14.2   Plastic Design of Beam-Columns

Beam-columns designed on the basis of plastic analysis shall satisfy the following interaction equations
for stability (Equation 3.138) and for strength (Equation 3.139).

$$\frac{P_u}{P_{cr}} + \frac{C_m M_u}{\left(1 - \frac{P_u}{P_e}\right) M_m} \leq 1.0 \qquad (3.138)$$

$$\frac{P_u}{P_y} + \frac{M_u}{1.18 M_p} \leq 1.0 \qquad (3.139)$$

where

P_u =   factored axial load
P_{cr} = $1.7 F_a A$, F_a is defined in Equation 3.16 and A is the cross-sectional area
P_y =   yield load $= A F_y$
P_e =   Euler buckling load $= \pi^2 E I/(Kl)^2$
C_m =   coefficient defined in the section on Compression Members
M_u =   factored moment
M_p =   plastic moment $= Z F_y$
M_m =   maximum moment that can be resisted by the member in the absence of axial load
      = M_{px} if the member is braced in the weak direction
      = $\{1.07 - [(l/r_y)\sqrt{F_y}]/3160\} M_{px} \leq M_{px}$ if the member is unbraced in the weak direction
l =   unbraced length of the member
r_y =   radius of gyration about the minor axis
M_{px} =   plastic moment about the major axis $= Z_x F_y$
F_y =   minimum specified yield stress

3.15   Defining Terms

ASD: Acronym for Allowable Stress Design.

Beamxcolumns: Structural members whose primary function is to carry loads both along and
      transverse to their longitudinal axes.

Biaxial bending: Simultaneous bending of a member about two orthogonal axes of the cross-
      section.

Builtxup members: Structural members made of structural elements jointed together by bolts,
      welds, or rivets.

Composite members: Structural members made of both steel and concrete.

Compression members: Structural members whose primary function is to carry loads along
      their longitudinal axes

Design strength: Resistance provided by the structural member obtained by multiplying the
      nominal strength of the member by a resistance factor.

Drift: Lateral deflection of a building.

Factored load: The product of the nominal load and a load factor.

Flexural members: Structural members whose primary function is to carry loads transverse to their longitudinal axes.

Limit state: A condition in which a structural or structural component becomes unsafe (strength limit state) or unfit for its intended function (serviceability limit state).

Load factor: A factor to account for the unavoidable deviations of the actual load from its nominal value and uncertainties in structural analysis in transforming the applied load into a load effect (axial force, shear, moment, etc.)

LRFD: Acronym for Load and Resistance Factor Design.

PD: Acronym for Plastic Design.

Plastic hinge: A yielded zone of a structural member in which the internal moment is equal to the plastic moment of the cross-section.

Resistance factor: A factor to account for the unavoidable deviations of the actual resistance of a member from its nominal value.

Service load: Nominal load expected to be supported by the structure or structural component under normal usage.

Sidesway inhibited frames: Frames in which lateral deflections are prevented by a system of bracing.

Sidesway uninhibited frames: Frames in which lateral deflections are not prevented by a system of bracing.

Shear lag: The phenomenon in which the stiffer (or more rigid) regions of a structure or structural component attract more stresses than the more flexible regions of the structure or structural component. Shear lag causes stresses to be unevenly distributed over the cross-section of the structure or structural component.

Tension field action: Post-buckling shear strength developed in the web of a plate girder. Tension field action can develop only if sufficient transverse stiffeners are provided to allow the girder to carry the applied load using truss-type action after the web has buckled.

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