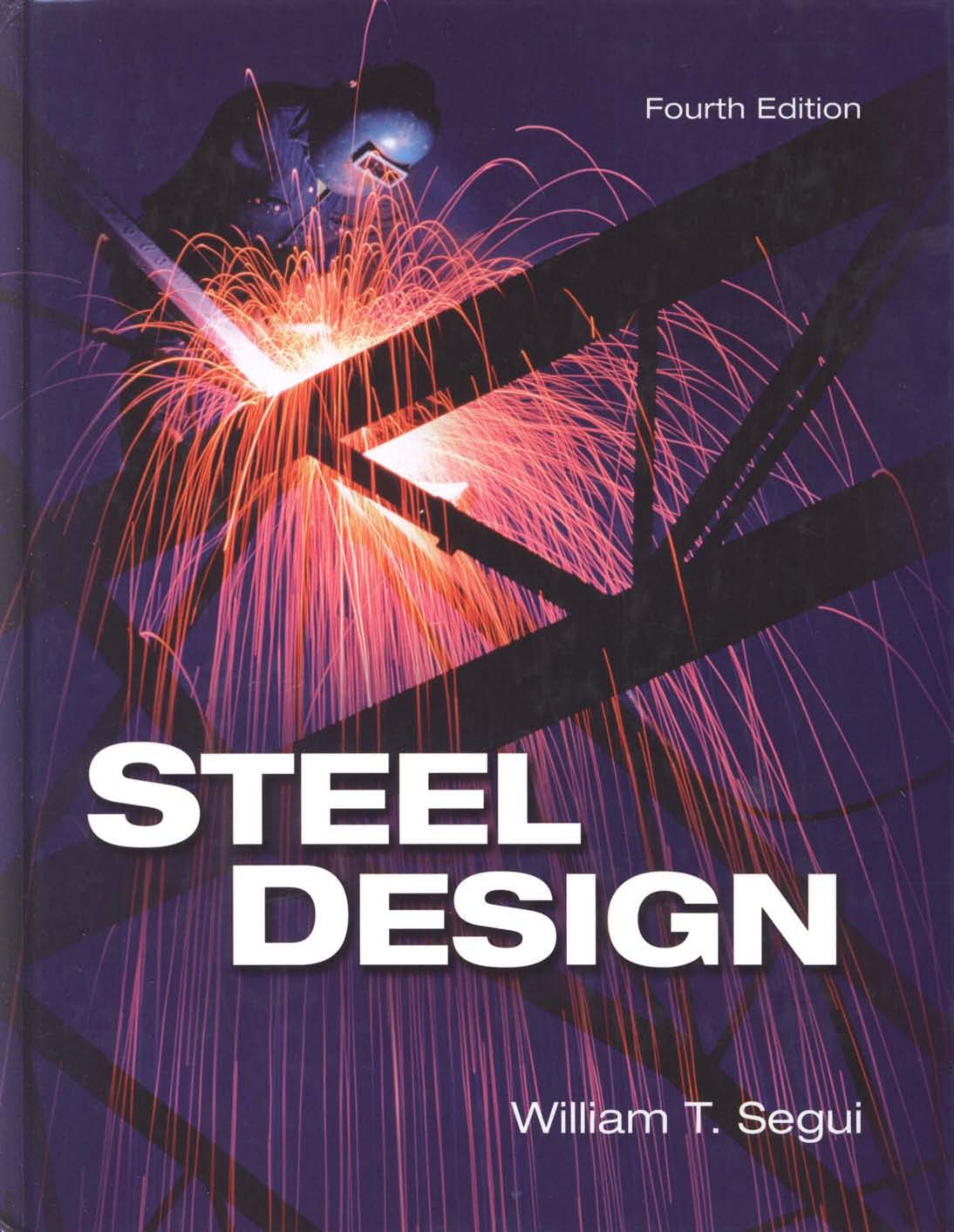
Fourth Edition
STEEL
DESIGN
William T. Segui

SYMBOLS

a moment arm for plastic moment internal couple, longitudinal spacing of connectors in a built-up compression member, distance from support to load, depth of equivalent compressive stress distribution in concrete at ultimate load, clear spacing of intermediate web stiffeners in a plate girder

a_w constant used in computing plate girder strength reduction factor

A area

A_1 bearing area of a bearing plate or base plate

A_2 full area of support for a bearing plate or base plate

A_b cross-sectional area of the unthreaded part of a bolt

A_c area of concrete flange in a composite beam, area of concrete in a composite column

A_e effective area

A_{eff} reduced area for sections with slender stiffened compression elements

A_f flange area

A_{fg} gross area of flange

A_{fn} net area of flange

A_g gross area

A_{gv} gross area in shear for block shear computation

A_n net area

A_{nt} net area in tension for block shear computation

A_{nv} net area in shear for block shear computation

A_{pb} bearing area of a plate girder stiffener

A_r area of reinforcing steel within the effective width of a composite beam slab

A_s area of steel cross section

A_{sc} cross-sectional area of a stud shear connector

A_{sr} area of longitudinal reinforcing steel in a composite column

A_{st} cross-sectional area of a stiffener

A_w web area

b width of a plate, width of cross-sectional element to be used in width-thickness ratio, effective flange width of a composite beam

b_b width of a beam flange or flange plate

b_e effective width of a slender stiffened compression element

b_f flange width

b_x x-axis flexural strength coefficient for beam-column design

b_y y-axis flexural strength coefficient for beam-column design

B width of an HSS, width of bearing plate or base plate, factor used in computing bending strength of double-angle and tee shapes

B_1, B_2 amplification factors for beam-columns

B_c bolt tensile force (including effects of prying)

c distance from elastic neutral axis to extreme fiber in bending, constant in equation for critical lateral-torsional buckling stress

C compressive force in an internal resisting couple

C_1 coefficient in equation for effective flexural rigidity of an encased composite column

C_2 coefficient in equation for compressive yield load, P_o, for a filled composite column

C_3 coefficient in equation for effective flexural rigidity of a filled composite column

C_b moment gradient factor for lateral-torsional buckling strength

C_m bending factor for beam-columns

C_v ratio of critical web shear stress to web shear yield stress in a plate girder

C_w warping constant

d total depth of a rolled steel shape, distance between axes (for use in the parallel axis theorem), bolt diameter

d' diameter of a staggered bolt

d_b beam depth, bolt diameter

d_c column depth

D service dead load effect to be used in computation of factored load combinations, outer diameter of a hollow circular steel shape, fillet weld size in sixteenths of an inch

D_s factor in equation for required intermediate stiffener area for a plate girder

D_u ratio of mean actual bolt pretension to specified minimum pretension

e eccentricity of load in a connection

E modulus of elasticity (29,000 ksi for structural steel), service earthquake load effect to be used in computation of factored load combinations

E_c modulus of elasticity of concrete

E_s modulus of elasticity of structural steel = 29,000 ksi

E_t tangent modulus of elasticity

f stress

f_1 direct shear stress in an eccentric welded shear connection

f_2 torsional shear stress in an eccentric welded shear connection

f_a axial compressive stress

f_b flexural stress

f_c flexural stress in concrete

f'_c 28-day compressive strength of concrete

f_p bearing stress

f_{sb} flexural stress at bottom of steel shape

f_{st} flexural stress at top of steel shape

f_t tensile stress

f_v shearing stress

F'_{nt} nominal bolt tensile strength (stress) in the presence of shear

F_a allowable axial compressive stress

F_{BM} shear strength (stress) of base metal in a welded connection

F_{cr} critical compressive or bending stress used to determine nominal strength

F_{cry} flexural buckling strength corresponding to the axis of symmetry in a structural tee or double-angle compression member

F_{crz} stress used in computing torsional or flexural-torsional buckling strength of a structural tee or double-angle compression member

F_e Euler buckling stress, critical elastic buckling stress in an unsymmetrical compression member (torsional or flexural-torsional buckling stress)

F_{ex}, F_{ey}, F_{ez} stresses used in computing torsional or flexural-torsional buckling strength

F_n nominal bolt shear or tensile strength (stress)

F_{nt} nominal bolt tensile strength (stress)

F_{nv} nominal bolt shear strength (stress)

F_{pl} stress at proportional limit

F_t allowable member tensile stress, ultimate tensile stress of a bolt, allowable bolt tensile stress

F_u ultimate tensile stress

F_v allowable member shear stress

F_W ultimate shearing stress of weld electrode

F_y yield stress

F_{yf}, F_{yw} yield stresses of flange and web

F_{yr} yield stress of reinforcing steel

F_{yst} yield stress of a stiffener

F_{yt} yield strength of tension flange

g gage distance for bolts (transverse spacing)

G shear modulus of elasticity = 11,200 ksi for structural steel

G_A, G_B factors for use in nomographs for effective length factor K

h width of web from toe of flange fillet to toe of flange fillet for a rolled shape, width of web from inside of flange to inside of flange for a welded shape, bolt hole diameter

h_c twice the distance from the elastic neutral axis to the inside face of the compression flange of a built-up flexural member (same as h for girders with equal flanges)

h_o distance between W-shape flange centroids

h_p twice the distance from the plastic neutral axis to the inside face of the compression flange of a built-up flexural member (same as h for girders with equal flanges)

h_{sc} hole factor for slip-critical bolts

H depth of an HSS, factor used in computation of flexural-torsional buckling strength of compression members, horizontal building loads, flange force in a moment connection

I moment of inertia (second moment of area)

$\bar{I}$ moment of inertia of component area about its centroidal axis

I_c moment of inertia of a column cross section

I_{eff} effective transformed moment of inertia of a partially composite beam

I_g moment of inertia of a girder cross section

I_{LB} lower-bound moment of inertia of a composite beam

I_s moment of inertia of steel section

I_{st} moment of inertia of a stiffener cross section

I_{tr} moment of inertia of transformed section

I_x, I_y moments of inertia about x and y axes

j constant used in computing required moment of inertia of a plate girder stiffener

J torsional constant, polar moment of inertia

k distance from outer face of flange to toe of fillet in the web of a rolled shape

k_c factor used in computing the flexural strength of a plate girder

k_s multiplier for bolt slip-critical strength when tension is present

k_v factor used in computing shear strength

K effective length factor for compression members

K_x, K_y, K_z effective length factors for x, y, and z axes

K_xL, K_yL, K_zL effective lengths for buckling about x, y, and z axes

ℓ length of a connection, length of end welds, factor for computing column base plate thickness, largest unbraced length of the flange of a plate girder

L service live load effect to be used in computation of factored load combinations, member length, story height, length of a weld segment

L_b unbraced beam length, unbraced length of a column in the equation for required bracing stiffness

L_c column length, distance from edge of bolt hole to edge of connected part or to edge of adjacent hole

L_g length of girder

L_p largest unbraced beam length for which lateral-torsional buckling will not occur

L_{pd} largest unbraced beam length for which plastic analysis can be used.

L_r unbraced beam length at which elastic lateral-torsional buckling will occur, service roof live load effect to be used in computation of factored load combinations

m length of unit width of plate in bending (for beam bearing plate and column base plate design)

M bending moment

Steel Design

Fourth Edition

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The University of Memphis

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by William T. Segui

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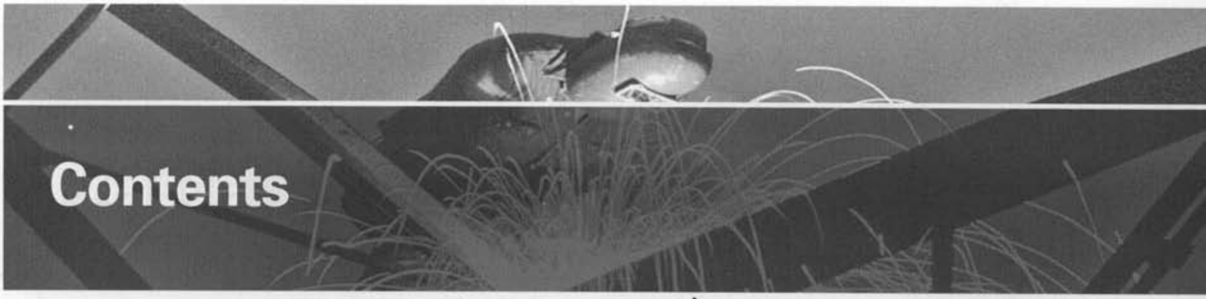

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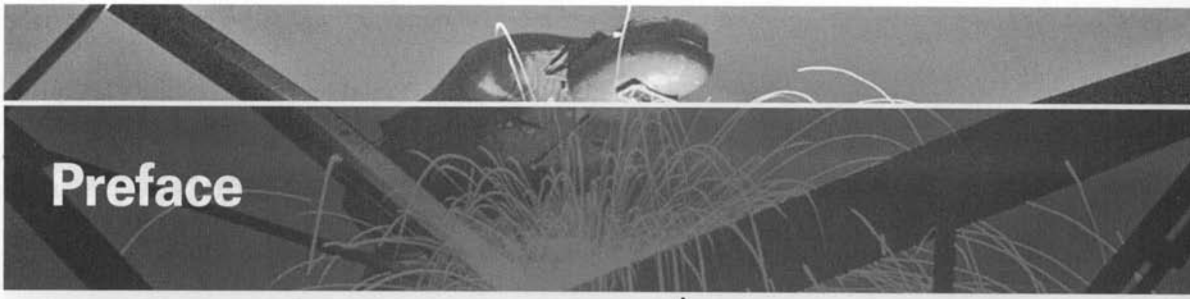


Preface

Steel Design, Fourth Edition covers the fundamentals of structural steel design. The emphasis is on the design of members and their connections rather than the integrated design of buildings. This book is intended for junior- and senior-level engineering students, although some of the later chapters can be used in graduate courses. Practicing civil engineers who need a review of current practice and the current AISC Specification and Manual will find the book useful as a reference. Students should have a background in mechanics of materials and analysis of statically determinate structures.

Steel Design is a revision of *LRFD Steel Design,* but because of the nature of the 2005 Specification and Manual of the American Institute of Steel Construction (AISC), it is more than a new edition of *LRFD Steel Design,* hence the change in title. Prior to the 2005 AISC documents, load and resistance factor design (LRFD) was covered by the 1999 AISC Specification and *LRFD Manual of Steel Construction, Third Edition.* Allowable stress design (ASD) was covered by the 1978 AISC Specification and *Manual of Steel Construction, Ninth Edition.* In 2005, the two approaches were unified in a single specification and a single manual, the thirteenth edition of the *Steel Construction Manual.* In addition, changes were made to many provisions of the specification, both in form and substance.

Both LRFD and ASD are covered in this textbook, but the emphasis is on LRFD. In most examples, both LRFD and ASD solutions are given. In those examples, the LRFD solution is given first. In some cases, the ASD solution is abbreviated but complete and independent of the LRFD solution. This usually involves some duplication, but is necessary if a reader is interested in only the ASD solution. In some ASD solutions where there would be a lengthy duplication, the reader is referred to the LRFD solution for that portion. In some of the examples, particularly in the later chapters, only an LRFD solution is given.

This book is designed so that an instructor can easily teach either LRFD or ASD. If time permits, both can be covered. One possibility is to cover the requirements for both but use mostly LRFD examples as the course progresses. As will be seen, the differences in the two approaches are mainly conceptual, and there is very little difference in the computations.

It is essential that students have a copy of the *Steel Construction Manual.* In order to promote familiarity with it, material from the *Manual* is not reproduced in this book so that the reader will be required to refer to the *Manual.* All notation in *Steel Design* is consistent with that in the *Manual* and AISC equation numbers are used along with sequential numbering of other equations according to the textbook chapter.

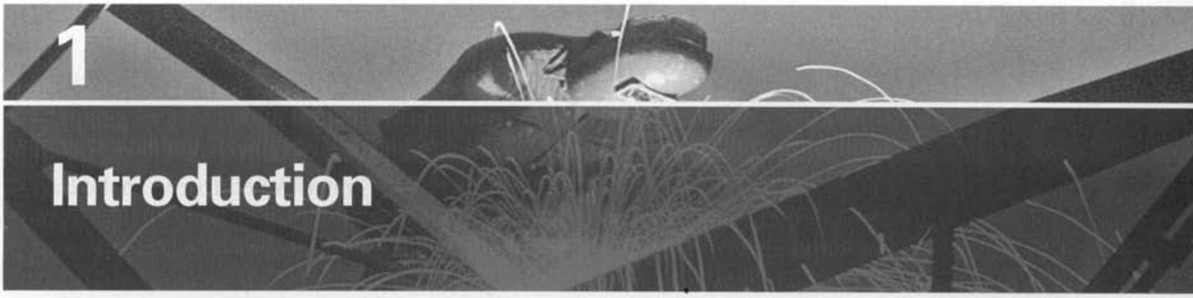
U.S. customary units are used throughout with no introduction of SI units. Although the *AISC Specification* now uses a dual system of units, the steel construction industry is still in a period of transition.

As far as design procedures are concerned, the application of fundamental principles is encouraged. Although this book is oriented toward practical design, sufficient theory is included to avoid a "cookbook" approach. Direct design methods are used where feasible, but no complicated design formulas have been developed. Instead, trial and error, with "educated guesses," is the rule. Tables, curves, and other design aids from the *Manual* are used, but they have a role that is subordinate to the use of basic equations. Assigned problems provide practice with both approaches and, where appropriate, the required approach is specified in the statement of the problem. In keeping with the objective of providing a basic textbook, a large number of assigned problems are given at the end of each chapter. Answers to selected problems are given at the back of the book, and an Instructor's Manual with solutions is available.

I would like to express my appreciation to Christopher Carson, General Manager of Thomson Engineering; Hilda Gowans, Developmental Editor for Thompson Learning; and Rose Kernan of RPK Editorial Services for their help during the production of this book. In addition, Christopher Hewitt of the American Institute of Steel Construction was very helpful in providing updates on the AISC Specification and Manual revisions as well as other assistance. Finally, I want to thank my wife, Angela, for her encouragement and for her valuable suggestions and assistance in proofreading the manuscript of this book.

I would appreciate learning of any errors that users of this book discover. I can be contacted at wsegui@memphis.edu.

William T. Segui



1 Introduction

1.1 STRUCTURAL DESIGN

The structural design of buildings, whether of structural steel or reinforced concrete, requires the determination of the overall proportions and dimensions of the supporting framework and the selection of the cross sections of individual members. In most cases the functional design, including the establishment of the number of stories and the floor plan, will have been done by an architect, and the structural engineer must work within the constraints imposed by this design. Ideally, the engineer and architect will collaborate throughout the design process to complete the project in an efficient manner. In effect, however, the design can be summed up as follows: The architect decides how the building should look; the engineer must make sure that it doesn't fall down. Although this distinction is an oversimplification, it affirms the first priority of the structural engineer: safety. Other important considerations include serviceability (how well the structure performs in terms of appearance and deflection) and economy. An economical structure requires an efficient use of materials and construction labor. Although this objective can usually be accomplished by a design that requires a minimum amount of material, savings can often be realized by using more material if it results in a simpler, more easily constructed project. In fact, materials account for a relatively small portion of the cost of a typical steel structure as compared with labor and other costs (Cross, 2005).

A good design requires the evaluation of several framing plans — that is, different arrangements of members and their connections. In other words, several alternative designs should be prepared and their costs compared. For each framing plan investigated, the individual components must be designed. To do so requires the structural analysis of the building frames and the computation of forces and bending moments in the individual members. Armed with this information, the structural designer can then select the appropriate cross section. Before any analysis, however, a decision must be made on the primary building material to be used; it will usually be reinforced concrete, structural steel, or both. Ideally, alternative designs should be prepared with each.

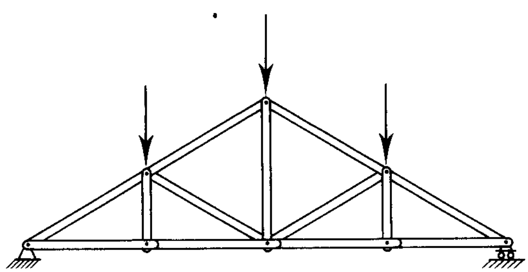
The emphasis in this book will be on the design of individual structural steel members and their connections. The structural engineer must select and evaluate the overall structural system in order to produce an efficient and economical design but cannot do so without a thorough understanding of the design of the components (the "building blocks") of the structure. Thus component design is the focus of this book.

Before discussing structural steel, we need to examine various types of structural members. Figure 1.1 shows a truss with vertical concentrated forces applied at the joints along the top chord. In keeping with the usual assumptions of truss analysis — pinned connections and loads applied only at the joints — each component of the truss will be a two-force member, subject to either axial compression or tension. For simply supported trusses loaded as shown — a typical loading condition — each of the top chord members will be in compression, and the bottom chord members will be in tension. The web members will either be in tension or compression, depending on their location and orientation and on the location of the loads.

Other types of members can be illustrated with the rigid frame of Figure 1.2a. The members of this frame are rigidly connected by welding and can be assumed to form a continuous structure. At the supports, the members are welded to a rectangular plate that is bolted to a concrete footing. Placing several of these frames in parallel and connecting them with additional members that are then covered with roofing material and walls produces a typical building system. Many important details have not been mentioned, but many small commercial buildings are constructed essentially in this manner. The design and analysis of each frame in the system begins with the idealization of the frame as a two-dimensional structure, as shown in Figure 1.2b. Because the frame has a plane of symmetry parallel to the page, we are able to treat the frame as two-dimensional and represent the frame members by their centerlines. (Although it is not shown in Figure 1.1, this same idealization is made with trusses, and the members are usually represented by their centerlines.) Note that the supports are represented as hinges (pins), not as fixed supports. If there is a possibility that the footing will undergo a slight rotation, or if the connection is flexible enough to allow a slight rotation, the support must be considered to be pinned. One assumption made in the usual methods of structural analysis is that deformations are very small, which means that only a slight rotation of the support is needed to qualify it as a pinned connection.

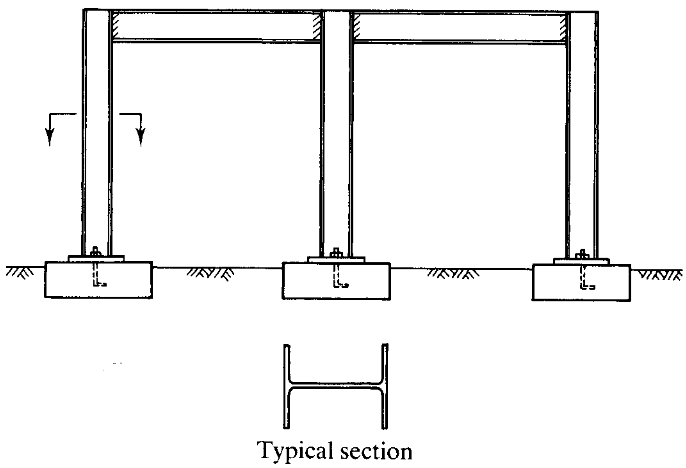
Once the geometry and support conditions of the idealized frame have been established, the loading must be determined. This determination usually involves apportioning a share of the total load to each frame. If the hypothetical structure under consideration is subjected to a uniformly distributed roof load, the portion carried by one frame will be a uniformly distributed line load measured in force per unit length, as shown in Figure 1.2b. Typical units would be kips per foot.

FIGURE 1.1

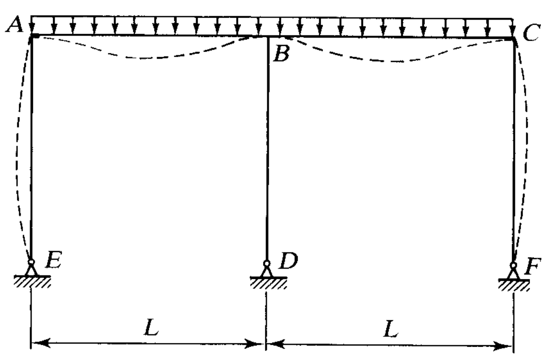


For the loading shown in Figure 1.2b, the frame will deform as indicated by the dashed line (drawn to a greatly exaggerated scale). The individual members of the frame can be classified according to the type of behavior represented by this deformed shape. The horizontal members *AB* and *BC* are subjected primarily to bending, or flexure, and are called *beams*. The vertical member *BD* is subjected to couples transferred from each beam, but for the symmetrical frame shown, they are equal and opposite, thereby canceling each other. Thus member *BD* is subjected only to axial compression arising from the vertical loads. In buildings, vertical compression members such as these are referred to as *columns*. The other two vertical members, *AE* and *CF*, must resist not only axial compression from the vertical loads but also a significant amount of bending. Such members are called *beam-columns*. In reality, all members, even those classified as beams or columns, will be subjected to both bending and axial load, but in many cases, the effects are minor and can be neglected.

FIGURE 1.2



(a)



(b)

In addition to the members described, this book covers the design of connections and the following special members: composite beams, composite columns, and plate girders.

1.2 LOADS

The forces that act on a structure are called *loads*. They belong to one of two broad categories: *dead load* and *live load*. Dead loads are those that are permanent, including the weight of the structure itself, which is sometimes called the *self-weight*. In addition to the weight of the structure, dead loads in a building include the weight of nonstructural components such as floor coverings, partitions, and suspended ceilings (with light fixtures, mechanical equipment, and plumbing). All of the loads mentioned thus far are forces resulting from gravity and are referred to as *gravity loads*. Live loads, which can also be gravity loads, are those that are not as permanent as dead loads. They may or may not be acting on the structure at any given time, and the location may not be fixed. Examples of live loads include furniture, equipment, and occupants of buildings. In general, the magnitude of a live load is not as well defined as that of a dead load, and it usually must be estimated. In many cases, a structural member must be investigated for various positions of a live load so that a potential failure condition is not overlooked.

If a live load is applied slowly and is not removed and reapplied an excessive number of times, the structure can be analyzed as if the load were static. If the load is applied suddenly, as would be the case when the structure supports a moving crane, the effects of impact must be accounted for. If the load is applied and removed many times over the life of the structure, fatigue stress becomes a problem, and its effects must be accounted for. Impact loading occurs in relatively few buildings, notably industrial buildings, and fatigue loading is rare, with thousands of load cycles over the life of the structure required before fatigue becomes a problem. For these reasons, all loading conditions in this book will be treated as static, and fatigue will not be considered.

Wind exerts a pressure or suction on the exterior surfaces of a building, and because of its transient nature, it properly belongs in the category of live loads. Because of the relative complexity of determining wind loads, however, wind is usually considered a separate category of loading. Because lateral loads are most detrimental to tall structures, wind loads are usually not as important for low buildings, but uplift on light roof systems can be critical. Although wind is present most of the time, wind loads of the magnitude considered in design are infrequent and are not considered to be fatigue loads.

Earthquake loads are another special category and need to be considered only in those geographic locations where there is a reasonable probability of occurrence. A structural analysis of the effects of an earthquake requires an analysis of the structure's response to the ground motion produced by the earthquake. Simpler methods are sometimes used in which the effects of the earthquake are simulated by a system of horizontal loads, similar to those resulting from wind pressure, acting at each floor level of the building.

Snow is another live load that is treated as a separate category. Adding to the uncertainty of this load is the complication of drift, which can cause much of the load to accumulate over a relatively small area.

Other types of live load are often treated as separate categories, such as hydrostatic pressure and soil pressure, but the cases we have enumerated are the ones ordinarily encountered in the design of structural steel building frames and their members.

1.3 BUILDING CODES

Buildings must be designed and constructed according to the provisions of a building code, which is a legal document containing requirements related to such things as structural safety, fire safety, plumbing, ventilation, and accessibility to the physically disabled. A building code has the force of law and is administered by a governmental entity such as a city, a county, or, for some large metropolitan areas, a consolidated government. Building codes do not give design procedures, but they do specify the design requirements and constraints that must be satisfied. Of particular importance to the structural engineer is the prescription of minimum live loads for buildings. Although the engineer is encouraged to investigate the actual loading conditions and attempt to determine realistic values, the structure must be able to support these specified minimum loads.

Although some large cities have their own building codes, many municipalities will modify a "model" building code to suit their particular needs and adopt it as modified. Model codes are written by various nonprofit organizations in a form that can be easily adopted by a governmental unit. Three national code organizations have developed model building codes: the *Uniform Building Code* (International Conference of Building Officials, 1999), the *Standard Building Code* (Southern Building Code Congress International, 1999), and the *BOCA National Building Code* (BOCA, 1999) (BOCA is an acronym for Building Officials and Code Administrators.) These codes have generally been used in different regions of the United States. The *Uniform Building Code* has been essentially the only one used west of the Mississippi, the *Standard Building Code* has been used in the southeastern states, and the *BOCA National Building Code* has been used in the northeastern part of the country.

A unified building code, the *International Building Code* (International Code Council, 2003), has been developed to eliminate some of the inconsistencies among the three national building codes. This was a joint effort by the three code organizations (ICBO, BOCA, and SBCCI). These organizations will continue to function, but the new code replaces the three regional codes.

Although it is not a building code, ASCE 7, *Minimum Design Loads for Buildings and Other Structures* (American Society of Civil Engineers, 2002) is similar in form to a building code. This standard provides load requirements in a format suitable for adoption as part of a code. The *International Building Code* incorporates much of ASCE 7 in its load provisions.

1.4 DESIGN SPECIFICATIONS

In contrast to building codes, design specifications give more specific guidance for the design of structural members and their connections. They present the guidelines and criteria that enable a structural engineer to achieve the objectives mandated by a building code. Design specifications represent what is considered to be good engineering practice based on the latest research. They are periodically revised and updated by the issuance of supplements or completely new editions. As with model building codes, design specifications are written in a legal format by nonprofit organizations. They have no legal standing on their own, but by presenting design criteria and limits in the form of legal mandates and prohibitions, they can easily be adopted, by reference, as part of a building code.

The specifications of most interest to the structural steel designer are those published by the following organizations.

1. **American Institute of Steel Construction (AISC):** This specification provides for the design of structural steel buildings and their connections. It is the one of primary concern in this book, and we discuss it in detail (AISC, 2005a).
2. **American Association of State Highway and Transportation Officials (AASHTO):** This specification covers the design of highway bridges and related structures. It provides for all structural materials normally used in bridges, including steel, reinforced concrete, and timber (AASHTO, 2002, 2004).
3. **American Railway Engineering and Maintenance-of-Way Association (AREMA):** The AREMA *Manual of Railway Engineering* covers the design of railway bridges and related structures (AREMA, 2005). This organization was formerly known as the American Railway Engineering Association (AREA).
4. **American Iron and Steel Institute (AISI):** This specification deals with cold-formed steel, which we discuss in Section 1.6 of this book (AISI, 2001).

1.5 STRUCTURAL STEEL

The earliest use of iron, the chief component of steel, was for small tools, in approximately 4000 B.C. (Murphy, 1957). This material was in the form of wrought iron, produced by heating ore in a charcoal fire. In the latter part of the eighteenth century and in the early nineteenth century, cast iron and wrought iron were used in various types of bridges. Steel, an alloy of primarily iron and carbon, with fewer impurities and less carbon than cast iron, was first used in heavy construction in the nineteenth century. With the advent of the Bessemer converter in 1855, steel began to displace wrought iron and cast iron in construction. In the United States, the first structural steel railroad bridge was the Eads bridge, constructed in 1874 in St. Louis, Missouri (Tall, 1964). In 1884, the first building with a steel frame was completed in Chicago.

The characteristics of steel that are of the most interest to structural engineers can be examined by plotting the results of a tensile test. If a test specimen is subjected to an axial load P, as shown in Figure 1.3a, the stress and strain can be computed as follows:

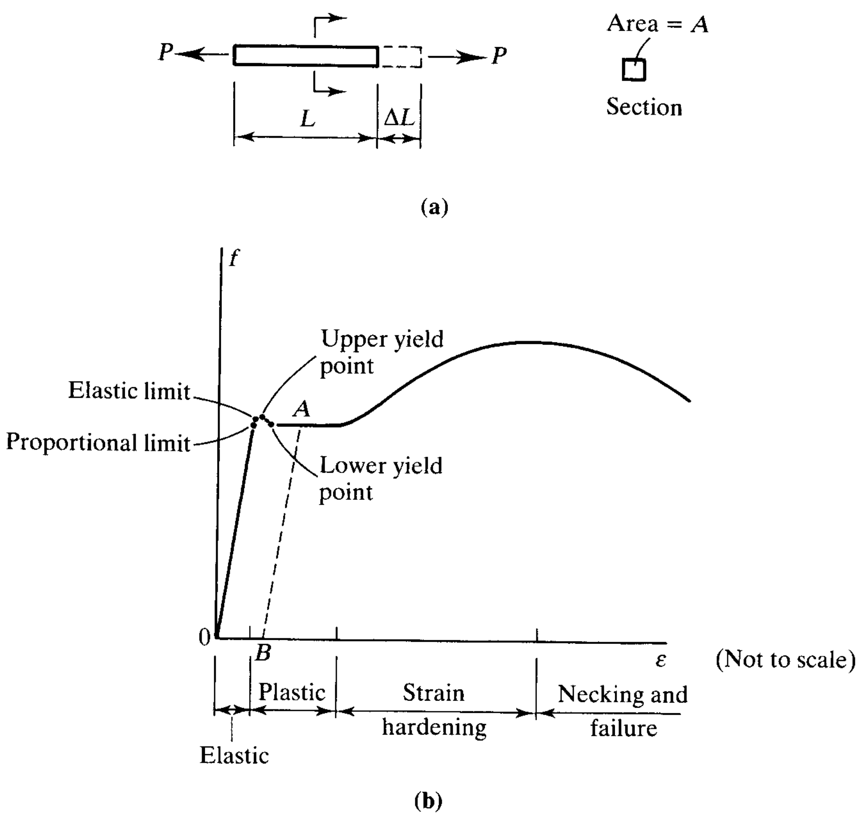
$$f = \frac{P}{A} \quad \text{and} \quad \varepsilon = \frac{\Delta L}{L}$$

where

f = axial tensile stress
A = cross-sectional area
ε = axial strain
L = length of specimen
ΔL = change in length

If the load is increased in increments from zero to the point of fracture, and stress and strain are computed at each step, a stress–strain curve such as the one shown in Figure 1.3b can be plotted. This curve is typical of a class of steel known as *ductile,* or *mild, steel.* The relationship between stress and strain is linear up to the proportional limit; the material is said to follow *Hooke's law.* A peak value, the upper yield point, is quickly reached after that, followed by a leveling off at the lower yield point.

FIGURE 1.3



The stress then remains constant, even though the strain continues to increase. At this stage of loading, the test specimen continues to elongate as long as the load is not removed, even though the load cannot be increased. This constant stress region is called the *yield plateau,* or *plastic range.* At a strain of approximately 12 times the strain at yield, strain hardening begins, and additional load (and stress) is required to cause additional elongation (and strain). A maximum value of stress is reached, after which the specimen begins to "neck down" as the stress decreases with increasing strain, and fracture occurs. Although the cross section is reduced during loading (the Poisson effect), the original cross-sectional area is used to compute all stresses. Stress computed in this way is known as *engineering stress.* If the original length is used to compute the strain, it is called *engineering strain.*

Steel exhibiting the behavior shown in Figure 1.3b is called *ductile* because of its ability to undergo large deformations before fracturing. Ductility can be measured by the elongation, defined as

$$e = \frac{L_f - L_0}{L_0} \times 100 \tag{1.1}$$

where

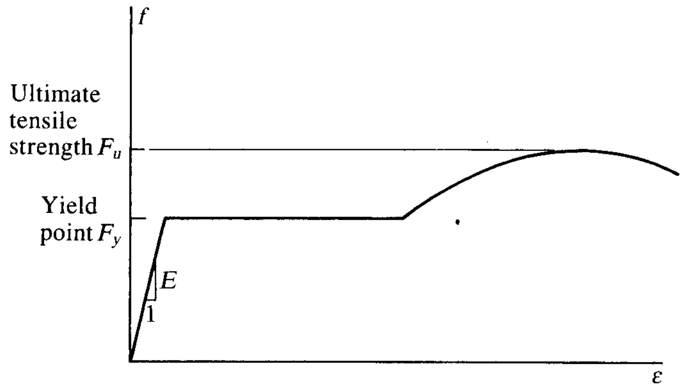
e = elongation (expressed as a percent)
L_f = length of the specimen at fracture
L_0 = original length

The elastic limit of the material is a stress that lies between the proportional limit and the upper yield point. Up to this stress, the specimen can be unloaded without permanent deformation; the unloading will be along the linear portion of the diagram, the same path followed during loading. This part of the stress–strain diagram is called the *elastic range.* Beyond the elastic limit, unloading will be along a straight line parallel to the initial linear part of the loading path, and there will be a permanent strain. For example, if the load is removed at point A in Figure 1.3b, the unloading will be along line AB, resulting in the permanent strain OB.

Figure 1.4 shows an idealized version of this stress–strain curve. The proportional limit, elastic limit, and the upper and lower yield points are all very close to one another and are treated as a single point called the *yield point,* defined by the stress F_y. The other point of interest to the structural engineer is the maximum value of stress that can be attained, called the *ultimate tensile strength,* F_u. The shape of this curve is typical of mild structural steels, which are different from one another primarily in the values of F_y and F_u. The ratio of stress to strain within the elastic range, denoted E and called *Young's modulus,* or *modulus of elasticity,* is the same for all structural steels and has a value of 29,000,000 psi (pounds per square inch) or 29,000 ksi (kips per square inch).

Figure 1.5 shows a typical stress–strain curve for high-strength steels, which are less ductile than the mild steels discussed thus far. Although there is a linear elastic portion and a distinct tensile strength, there is no well-defined yield point or yield plateau. To use these higher-strength steels in a manner consistent with the use of ductile steels, some value of stress must be chosen as a value for F_y so that the same procedures and formulas can be used with all structural steels. Although there is no yield

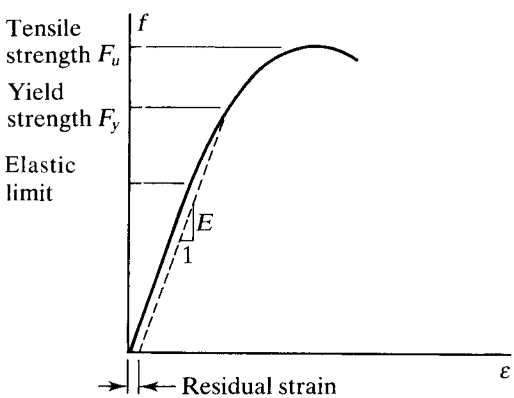
FIGURE 1.4



point, one needs to be defined. As previously shown, when a steel is stressed beyond its elastic limit and then unloaded, the path followed to zero stress will not be the original path from zero stress; it will be along a line having the slope of the linear portion of the path followed during loading — that is, a slope equal to E, the modulus of elasticity. Thus there will be a residual strain, or permanent set, after unloading. The yield stress for steel with a stress–strain curve of the type shown in Figure 1.5 is called the *yield strength* and is defined as the stress at the point of unloading that corresponds to a permanent strain of some arbitrarily defined amount. A strain of 0.002 is usually selected, and this method of determining the yield strength is called the *0.2% offset method.* As previously mentioned, the two properties usually needed in structural steel design are F_u and F_y, regardless of the shape of the stress–strain curve and regardless of how F_y was obtained. For this reason, the generic term *yield stress* is used, and it can mean either yield point or yield strength.

The various properties of structural steel, including strength and ductility, are determined by its chemical composition. Steel is an alloy, its principal component being iron. Another component of all structural steels, although in much smaller amounts, is carbon, which contributes to strength but reduces ductility. Other components of some grades of steel include copper, manganese, nickel, chromium, molybdenum, and silicon. Structural steels can be grouped according to their composition as follows.

FIGURE 1.5



1. **Plain carbon steels:** mostly iron and carbon, with less than 1% carbon.
2. **Low-alloy steels:** iron and carbon plus other components (usually less than 5%). The additional components are primarily for increasing strength, which is accomplished at the expense of a reduction in ductility.
3. **High-alloy or specialty steels:** similar in composition to the low-alloy steels but with a higher percentage of the components added to iron and carbon. These steels are higher in strength than the plain carbon steels and also have some special quality, such as resistance to corrosion.

Different grades of structural steel are identified by the designation assigned them by the American Society for Testing and Materials (ASTM). This organization develops standards for defining materials in terms of their composition, properties, and performance, and it prescribes specific tests for measuring these attributes (ASTM, 2005a). One of the most commonly used structural steels is a mild steel designated as ASTM A36, or A36 for short. It has a stress–strain curve of the type shown in Figures 1.3b and 1.4 and has the following tensile properties.

Yield stress: $F_y = 36{,}000$ psi (36 ksi)
Tensile strength: $F_u = 58{,}000$ psi to 80,000 psi (58 ksi to 80 ksi)

A36 steel is classified as a plain carbon steel, and it has the following components (other than iron).

Carbon:	0.26% (maximum)
Phosphorous:	0.04% (maximum)
Sulfur:	0.05% (maximum)

These percentages are approximate, the exact values depending on the form of the finished steel product. A36 is a ductile steel, with an elongation as defined by Equation 1.1 of 20% based on an undeformed original length of 8 inches.

Steel producers who provide A36 steel must certify that it meets the ASTM standard. The values for yield stress and tensile strength shown are minimum requirements; they may be exceeded and usually are to a certain extent. The tensile strength is given as a range of values because for A36 steel, this property cannot be achieved to the same degree of precision as the yield stress.

Other commonly used structural steels are ASTM A572 Grade 50 and ASTM A992. These two steels are very similar in both tensile properties and chemical composition, with a maximum carbon content of 0.23%. A comparison of the tensile properties of A36, A572 Grade 50, and A992 is given in Table 1.1.

TABLE 1.1

Property	A36	A572 Gr. 50	A992
Yield point, min.	36 ksi	50 ksi	50 ksi
Tensile strength, min.	58 to 80 ksi	65 ksi	65 ksi
Yield to tensile ratio, max.	—	—	0.85
Elongation in 8 in., min.	20%	18%	18%

1.6 STANDARD CROSS-SECTIONAL SHAPES

In the design process outlined earlier, one of the objectives — and the primary emphasis of this book — is the selection of the appropriate cross sections for the individual members of the structure being designed. Most often, this selection will entail choosing a standard cross-sectional shape that is widely available rather than requiring the fabrication of a shape with unique dimensions and properties. The selection of an "off-the-shelf" item will almost always be the most economical choice, even if it means using slightly more material. The largest category of standard shapes includes those produced by *hot-rolling*. In this manufacturing process, which takes place in a mill, molten steel is taken from the furnace and poured into a *continuous casting* system where the steel solidifies but is never allowed to cool completely. The hot steel passes through a series of rollers that squeeze the material into the desired cross-sectional shape. Rolling the steel while it is still hot allows it to be deformed with no resulting loss in ductility, as would be the case with cold-working. During the rolling process, the member increases in length and is cut to standard lengths, usually a maximum of 65 to 75 feet, which are subsequently cut (in a fabricating shop) to the lengths required for a particular structure.

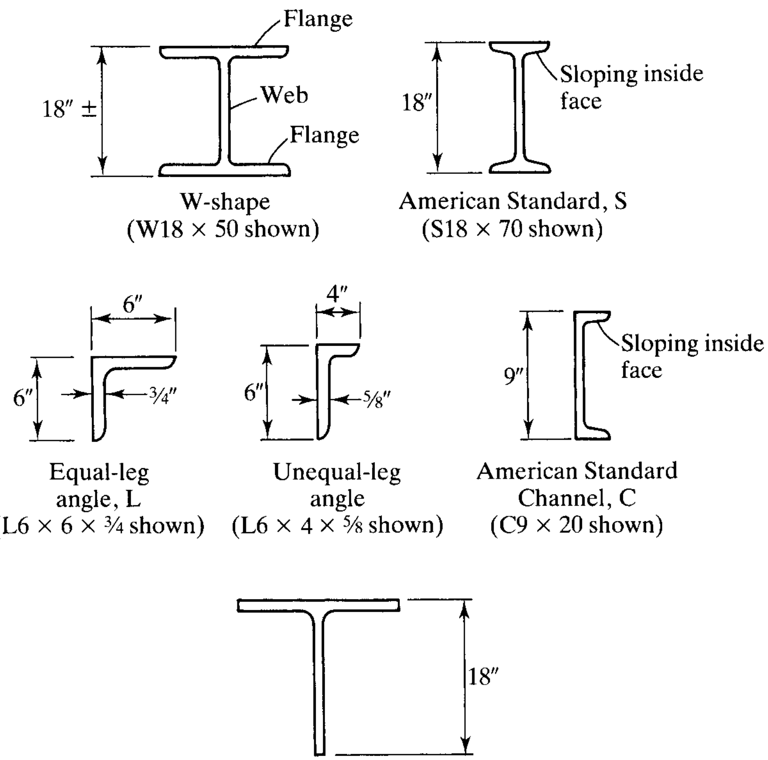
Cross sections of some of the more commonly used hot-rolled shapes are shown in Figure 1.6. The dimensions and designations of the standard available shapes are defined in the ASTM standards (ASTM, 2005b). The *W-shape,* also called a *wide-flange shape,* consists of two parallel flanges separated by a single web. The orientation of these elements is such that the cross section has two axes of symmetry. A typical designation would be "W18 × 50," where W indicates the type of shape, 18 is the nominal depth parallel to the web, and 50 is the weight in pounds per foot of length. The nominal depth is the approximate depth expressed in whole inches. For some of the lighter shapes, it is equal to the depth to the nearest inch, but this is not a general rule for the W-shapes. All of the W-shapes of a given nominal size can be grouped into families that have the same depth from inside-of-flange to inside-of-flange but with different flange thicknesses.

The *American Standard,* or *S-shape,* is similar to the W-shape in having two parallel flanges, a single web, and two axes of symmetry. The difference is in the proportions: The flanges of the W are wider in relation to the web than are the flanges of the S. In addition, the outside and inside faces of the flanges of the W-shape are parallel, whereas the inside faces of the flanges of the S-shape slope with respect to the outside faces. An example of the designation of an S-shape is "S18 × 70," with the S indicating the type of shape, and the two numbers giving the depth in inches and the weight in pounds per foot. This shape was formerly called an *I-beam.*

The angle shapes are available in either equal-leg or unequal-leg versions. A typical designation would be "L6 × 6 × ¾" or "L6 × 4 × ⅝." The three numbers are the lengths of each of the two legs as measured from the corner, or heel, to the toe at the other end of the leg, and the thickness, which is the same for both legs. In the case of the unequal-leg angle, the longer leg dimension is always given first. Although this designation provides all of the dimensions, it does not provide the weight per foot.

The *American Standard Channel,* or *C-shape,* has two flanges and a web, with only one axis of symmetry; it carries a designation such as "C9 × 20." This notation is

FIGURE 1.6



W-shape (W18 × 50 shown)

American Standard, S (S18 × 70 shown)

Equal-leg angle, L (L6 × 6 × 3/4 shown)

Unequal-leg angle (L6 × 4 × 5/8 shown)

American Standard Channel, C (C9 × 20 shown)

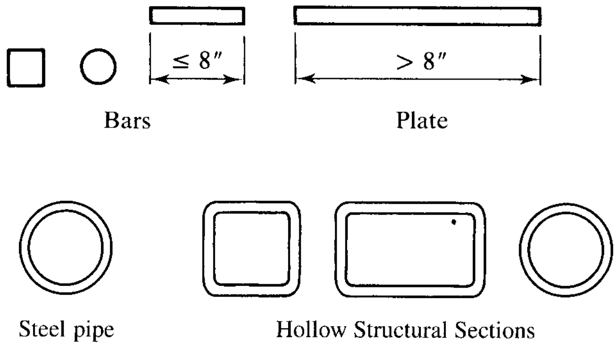
Structural Tee: WT, ST, or MT (WT18 × 115 shown)

similar to that for W- and S-shapes, with the first number giving the total depth in inches parallel to the web and the second number the weight in pounds per linear foot. For the channel, however, the depth is exact rather than nominal. The inside faces of the flanges are sloping, just as with the American Standard shape. Miscellaneous Channels — for example, the MC10 × 25 — are similar to American Standard Channels.

The *Structural Tee* is produced by splitting an I-shaped member at middepth. This shape is sometimes referred to as a *split-tee*. The prefix of the designation is either WT, ST, or MT, depending on which shape is the "parent." For example, a WT18 × 105 has a nominal depth of 18 inches and a weight of 105 pounds per foot, and is cut from a W36 × 210. Similarly, an ST10 × 33 is cut from an S20 × 66, and an MT5 × 4 is cut from an M10 × 8. The "M" is for "miscellaneous." The M-shape has two parallel flanges and a web, but it does not fit exactly into either the W or S categories. The HP shape, used for bearing piles, has parallel flange surfaces, approximately the same width and depth, and equal flange and web thicknesses. HP-shapes are designated in the same manner as the W-shape; for example, HP14 × 117.

Other frequently used cross-sectional shapes are shown in Figure 1.7. *Bars* can have circular, square, or rectangular cross sections. If the width of a rectangular shape is 8 inches or less, it is classified as a bar. If the width is more than 8 inches, the shape

FIGURE 1.7



is classified as a *plate*. The usual designation for both is the abbreviation PL (for plate, even though it could actually be a bar) followed by the thickness in inches, the width in inches, and the length in feet and inches; for example, PL $\frac{3}{8} \times 5 \times 3'\text{-}2\frac{1}{2}''$. Although plates and bars are available in increments of $\frac{1}{16}$ inch, it is customary to specify dimensions to the nearest $\frac{1}{8}$ inch. Bars and plates are formed by hot-rolling.

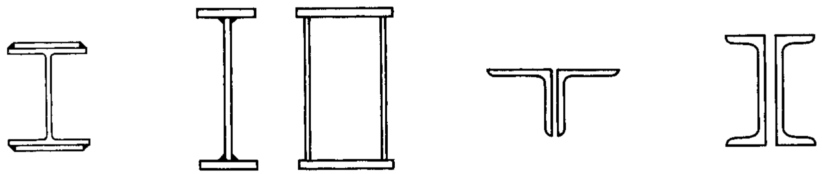
Also shown in Figure 1.7 are hollow shapes, which can be produced either by bending plate material into the desired shape and welding the seam or by hot-working to produce a seamless shape. Most hollow structural sections available in the United States today are produced by cold-forming and welding (Sherman, 1997). The shapes are categorized as steel pipe, round HSS, and square and rectangular HSS. The designation HSS is for "Hollow Structural Sections."

Steel pipe is available as standard, extra-strong, or double-extra-strong, with designations such as Pipe 5 Std., Pipe 5 x-strong, or Pipe 5 xx-strong, where 5 is the nominal outer diameter in inches. The different strengths correspond to different wall thicknesses for the same outer diameter. For nominal outer diameters greater than 12 inches, the designation is the outer diameter and wall thickness in inches, expressed to three decimal places; for example, Pipe 14.000×0.375.

Round HSS are designated by outer diameter and wall thickness, expressed to three decimal places; for example, HSS 8.625×0.250. Square and rectangular HSS are designated by nominal outside dimensions and wall thickness, expressed in rational numbers; for example, HSS $7 \times 5 \times \frac{3}{8}$.

Other shapes are available, but those just described are the ones most frequently used. In most cases, one of these standard shapes will satisfy design requirements. If the requirements are especially severe, then a built-up section, such as one of those shown in Figure 1.8, may be needed. Sometimes a standard shape is augmented by additional cross-sectional elements, as when a cover plate is welded to one or both flanges of a W-shape. Building up sections is an effective way of strengthening an existing structure that is being rehabilitated or modified for some use other than the one for which it was designed. Sometimes a built-up shape must be used because none of the standard rolled shapes are large enough; that is, the cross section does not have enough area or moment of inertia. In such cases, plate girders can be used. These can be I-shaped sections, with two flanges and a web, or box sections, with two flanges and two webs. The components can be welded together and can be designed to have exactly the properties needed. Built-up shapes can also be created by

FIGURE 1.8



attaching two or more standard rolled shapes to each other. A widely used combination is a pair of angles placed back-to-back and connected at intervals along their length. This is called a *double-angle shape*. Another combination is the double-channel shape (either American Standard or Miscellaneous Channel). There are many other possibilities, some of which we illustrate throughout this book.

The most commonly used steels for rolled shapes and plate material are ASTM A36, A572, and A992. ASTM A36 is usually specified for angles and plates; A36 or A572 Grade 50 for S, M, and channel shapes; A572 Grade 50 for HP shapes; and A992 for W shapes. (These three steels were compared in Table 1.1 in Section 1.5.) Steel pipe is available in ASTM A53 Grade B only. ASTM A500 is usually specified for hollow structural sections (HSS). These recommendations are summarized in Table 1.2. Other steels can be used for these shapes, but the ones listed in Table 1.2 are the most common (Carter, 2004).

Another category of steel products for structural applications is cold-formed steel. Structural shapes of this type are created by bending thin material such as sheet steel or plate into the desired shape without heating. Typical cross sections are shown in Figure 1.9. Only relatively thin material can be used, and the resulting shapes are suitable only for light applications. An advantage of this product is its versatility, since almost any conceivable cross-sectional shape can easily be formed. In addition, cold-working will increase the yield point of the steel, and under certain conditions it may be accounted for in design (AISI, 2001). This increase comes at the expense of a reduction in ductility, however. Because of the thinness of the cross-sectional elements, the problem of instability (discussed in Chapters 4 and 5) is a particularly important factor in the design of cold-formed steel structures.

TABLE 1.2

Shape	Preferred Steel
Angles	A36
Plates	A36
S, M, C, MC	A36 or A572 Grade 50
HP	A572 Grade 50
W	A992
Pipe	A53 Grade B (only choice)
HSS	A500 Grade B or C

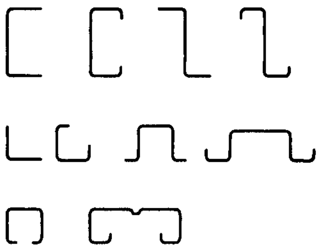


FIGURE 1.9

Problems

Note The following problems illustrate the concepts of stress and strain covered in Section 1.5. The materials cited in these problems are not necessarily steel.

1.5-1 A tensile test was performed on a metal specimen with a circular cross section. The diameter was measured to be 0.550 inch. Two marks were mode along the length of the specimen and were measured to be 2.030 inches apart. This distance is defined as the *gage length,* and all length measurements are made between the two marks. The specimen was loaded to failure. Fracture occurred at a load of 28,500 pounds. The specimen was then reassembled, and the diameter and gage length were measured to be 0.430 inch and 2.300 inches. Determine the

a. Ultimate tensile stress in ksi.

b. Elongation as a percentage.

c. Reduction in cross-sectional area as a percentage.

1.5-2 A tensile test was performed on a metal specimen having a circular cross section with a diameter of $\frac{1}{2}$ inch. The *gage length* (the length over which the elongation is measured) is 2 inches. For a load 13.5 kips, the elongation was 4.66×10^{-3} inches. If the load is assumed to be within the linear elastic range of the material, determine the modulus of elasticity.

1.5-3 A tensile test was performed on a metal specimen having a circular cross section with a diameter of 0.510 inch. For each increment of load applied, the strain was directly determined by means of a *strain gage* attached to the specimen. The results are shown in Table 1.5.1.

a. Prepare a table of stress and strain.

b. Plot these data to obtain a stress–strain curve. Do not connect the data points; draw a *best-fit* straight line through them.

c. Determine the modulus of elasticity as the slope of the best-fit line.

Load (lb)	Strain $\times 10^6$ (in./in.)
0	0
250	37.1
500	70.3
1000	129.1
1500	230.1
2000	259.4
2500	372.4
3000	457.7
3500	586.5

TABLE 1.5.1

1.5-4 A tensile test was performed on a metal specimen with a diameter of ½ inch and a gage length (the length over which the elongation is measured) of 4 inches. The data were plotted on a load-displacement graph, P vs. ΔL. A best-fit line was drawn through the points, and the slope of the straight-line portion was calculated to be $P/\Delta L = 1392$ kips/in. What is the modulus of elasticity?

1.5-5 The results of a tensile test are shown in Table 1.5.2. The test was performed on a metal specimen with a circular cross section. The diameter was ⅜ inch and the gage length (the length over which the elongation is measured) was 2 inches.

a. Use the data in Table 1.5.2 to produce a table of stress and strain values.

b. Plot the stress–strain data and draw a best-fit curve.

c. Compute the modulus of elasticity from the initial slope of the curve.

d. Estimate the yield stress.

Load (lb)	Elongation $\times 10^6$ (in.)
0	0
550	350
1100	700
1700	900
2200	1350
2800	1760
3300	2200
3900	2460
4400	2860
4900	3800
4970	5300
5025	7800

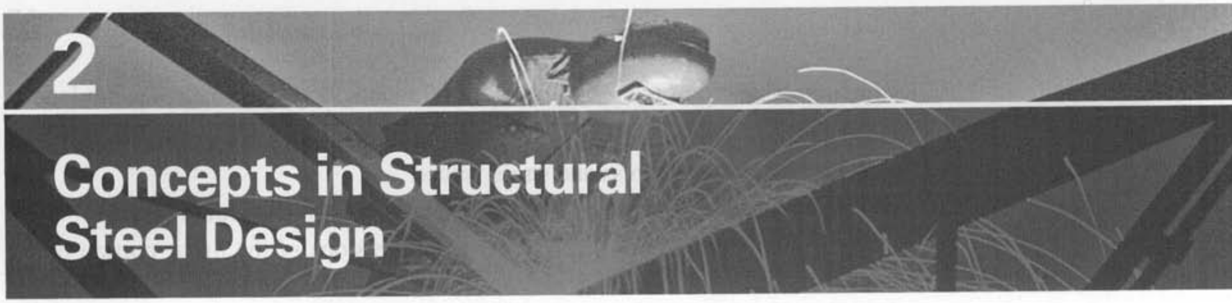
TABLE 1.5.2

1.5-6 The data in Table 1.5.3 were obtained from a tensile test of a metal specimen with a rectangular cross section of 0.2011 in.^{2} in area and a gage length (the length over which the elongation is measured) of 2.000 inches. The specimen was not loaded to failure.

a. Generate a table of stress and strain values.
b. Plot these values and draw a best-fit line to obtain a stress–strain curve.
c. Determine the modulus of elasticity from the slope of the linear portion of the curve.
d. Estimate the value of the proportional limit.
e. Use the 0.2% offset method to determine the yield stress.

Load (kips)	Elongation $\times 10^3$ (in.)
0	0
1	0.160
2	0.352
3	0.706
4	1.012
5	1.434
6	1.712
7	1.986
8	2.286
9	2.612
10	2.938
11	3.274
12	3.632
13	3.976
14	4.386
15	4.640
16	4.988
17	5.432
18	5.862
19	6.362
20	7.304
21	8.072
22	9.044
23	11.310
24	14.120
25	20.044
26	29.106

TABLE 1.5.3



2

Concepts in Structural Steel Design

2.1 DESIGN PHILOSOPHIES

As discussed earlier, the design of a structural member entails the selection of a cross section that will safely and economically resist the applied loads. Economy usually means minimum weight — that is, the minimum amount of steel. This amount corresponds to the cross section with the smallest weight per foot, which is the one with the smallest cross-sectional area. Although other considerations, such as ease of construction, may ultimately affect the choice of member size, the process begins with the selection of the lightest cross-sectional shape that will do the job. Having established this objective, the engineer must decide how to do it safely, which is where different approaches to design come into play. The fundamental requirement of structural design is that the required strength not exceed the available strength; that is,

$$\text{required strength} \le \text{available strength}$$

In *allowable strength design* (ASD), a member is selected that has cross-sectional properties such as area and moment of inertia that are large enough to prevent the maximum applied axial force, shear, or bending moment from exceeding an allowable, or permissible, value. This allowable value is obtained by dividing the nominal, or theoretical, strength by a factor of safety. This can be expressed as

$$\text{required strength} \le \text{allowable strength} \tag{2.1}$$

where

$$\text{allowable strength} = \frac{\text{nominal strength}}{\text{safety factor}}$$

Strength can be an axial force strength (as in tension or compression members), a flexural strength (moment strength), or a shear strength.

If stresses are used instead of forces or moments, the relationship of Equation 2.1 becomes

$$\text{maximum applied stress} \le \text{allowable stress} \tag{2.2}$$

This approach is called *allowable stress design.* The allowable stress will be in the elastic range of the material (see Figure 1.3). This approach to design is also called *elastic design* or *working stress design.* Working stresses are those resulting from the working loads, which are the applied loads. Working loads are also known as *service* loads.

Plastic design is based on a consideration of failure conditions rather than working load conditions. A member is selected by using the criterion that the structure will fail at a load substantially higher than the working load. Failure in this context means either collapse or extremely large deformations. The term *plastic* is used because, at failure, parts of the member will be subjected to very large strains — large enough to put the member into the plastic range (see Figure 1.3b). When the entire cross section becomes plastic at enough locations, "plastic hinges" will form at those locations, creating a *collapse mechanism.* As the actual loads will be less than the failure loads by a factor of safety known as the *load factor,* members designed this way are not unsafe, despite being designed based on what happens at failure. This design procedure is roughly as follows.

1. Multiply the working loads (service loads) by the load factor to obtain the failure loads.
2. Determine the cross-sectional properties needed to resist failure under these loads. (A member with these properties is said to have sufficient strength and would be at the verge of failure when subjected to the factored loads.)
3. Select the lightest cross-sectional shape that has these properties.

Members designed by plastic theory would reach the point of failure under the factored loads but are safe under actual working loads.

Load and resistance factor design (LRFD) is similar to plastic design in that strength, or the failure condition, is considered. Load factors are applied to the service loads, and a member is selected that will have enough strength to resist the factored loads. In addition, the theoretical strength of the member is reduced by the application of a resistance factor. The criterion that must be satisfied in the selection of a member is

$$\text{Factored load} \leq \text{factored strength} \tag{2.3}$$

In this expression, the factored load is actually the sum of all service loads to be resisted by the member, each multiplied by its own load factor. For example, dead loads will have load factors that are different from those for live loads. The factored strength is the theoretical strength multiplied by a resistance factor. Equation 2.3 can therefore be written as

$$\sum (\text{Loads} \times \text{load factors}) \leq \text{resistance} \times \text{resistance factor} \tag{2.4}$$

The factored load is a failure load greater than the total actual service load, so the load factors are usually greater than unity. However, the factored strength is a reduced, usable strength, and the resistance factor is usually less than unity. The factored loads are the loads that bring the structure or member to its limit. In terms of safety, this *limit state* can be fracture, yielding, or buckling, and the factored resistance is the useful strength of the member, reduced from the theoretical value by the resistance factor. The limit state can also be one of serviceability, such as a maximum acceptable deflection.

2.2 AMERICAN INSTITUTE OF STEEL CONSTRUCTION SPECIFICATION

Because the emphasis of this book is on the design of structural steel building members and their connections, the Specification of the American Institute of Steel Construction is the design specification of most importance here. It is written and kept current by an AISC committee comprising structural engineering practitioners, educators, steel producers, and fabricators. New editions are published periodically, and supplements are issued when interim revisions are needed. Allowable stress design has been the primary method used for structural steel buildings since the first AISC Specification was issued in 1923, although plastic design was made part of the Specification in 1963. In 1986, AISC issued the first specification for load and resistance factor design along with a companion *Manual of Steel Construction.* The purpose of these two documents was to provide an alternative to allowable stress design, much as plastic design is an alternative. The current specification (AISC, 2005a) incorporates both LRFD and ASD.

The LRFD provisions are based on research reported in eight papers published in 1978 in the *Structural Journal of the American Society of Civil Engineers* (Ravindra and Galambos; Yura, Galambos, and Ravindra; Bjorhovde, Galambos, and Ravindra; Cooper, Galambos, and Ravindra; Hansell et al.; Fisher et al.; Ravindra, Cornell, and Galambos; Galambos and Ravindra, 1978).

Although load and resistance factor design was not introduced into the AISC Specification until 1986, it is not a recent concept; since 1974, it has been used in Canada, where it is known as *limit states design.* It is also the basis of most European building codes. In the United States, LRFD has been an accepted method of design for reinforced concrete for years and is the primary method authorized in the American Concrete Institute's Building Code, where it is known as *strength design* (ACI, 2005). Highway bridge design standards provide for both allowable stress design (AASHTO, 2002) and load and resistance factor design (AASHTO, 2004).

The AISC Specification is published as a stand-alone document, but it is also part of the *Steel Construction Manual,* which we discuss in the next section. Except for such specialized steel products as cold-formed steel, which is covered by a different specification (AISI, 2001), the AISC Specification is the standard by which virtually all structural steel buildings in this country are designed and constructed. Hence the student of structural steel design must have ready access to his document. The details of the Specification will be covered in the chapters that follow, but we discuss the overall organization here.

The Specification consists of three parts: the main body, the appendixes, and the Commentary. The body is alphabetically organized into Chapters A through M. Within each chapter, major headings are labeled with the chapter designation followed by a number. Further subdivisions are numerically labeled. For example, the types of structural steel authorized are listed in Chapter A, "General Provisions," under Section A3. Material, and, under it, Section 1. Structural Steel Materials. The main body of the Specification is followed by appendixes 1–7. The Appendix section is followed by the Commentary, which gives background and elaboration on many of the provisions of the Specification. Its organizational scheme is the same as that of the Specification, so material applicable to a particular section can be easily located.

The Specification incorporates both U.S. customary and metric (SI) units. Where possible, equations and expressions are expressed in non-dimensional form by leaving quantities such as yield stress and modulus of elasticity in symbolic form, thereby avoiding giving units. When this is not possible, U.S. customary units are given, followed by SI units in parentheses. Although there is a strong move to metrication in the steel industry, most structural design in the United States is still done in U.S. customary units, and this textbook uses only U.S. customary units.

2.3 LOAD FACTORS, RESISTANCE FACTORS, AND LOAD COMBINATIONS FOR LRFD

Equation 2.4 can be written more precisely as

$$\Sigma \gamma_i Q_i \leq \phi R_n \tag{2.5}$$

where

Q_i = a load effect (a force or a moment)
γ_i = a load factor
R_n = the nominal resistance, or strength, of the component under consideration
ϕ = resistance factor

The factored resistance ϕR_n is called the *design strength.* The summation on the left side of Equation 2.5 is over the total number of load effects (including, but not limited to, dead load and live load), where each load effect can be associated with a different load factor. Not only can each load effect have a different load factor but also the value of the load factor for a particular load effect will depend on the combination of loads under consideration. Equation 2.5 can also be written in the form

$$R_u \leq \phi R_n \tag{2.6}$$

where

R_u = required strength = sum of factored load effects (forces or moments)

Section B2 of the AISC Specification requires that the load factors and load combinations given in ASCE 7 (ASCE 2002) be used. These load factors and load combinations are based on extensive statistical studies. The seven combinations are as follows:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W)$
4. $1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + 0.5L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

where

D = dead load
E = earthquake load
F = load due to fluids with well-defined pressures and maximum heights
H = load due to lateral earth pressure, groundwater pressure, or pressure of bulk materials
L = live load
L_r = roof live load
R = rain load*
S = snow load
T = self-straining force
W = wind load

(If a governing building code specifies other load combinations, then they should be used.)

Normally, fluid pressure F, earth pressure H, and self-straining force T are not applicable to the design of structural steel members, and we will omit them from this point forward. In addition, combinations 6 and 7 can be combined. With these and one other slight modification, the list of required load combinations becomes

$$1.4D \tag{1}$$

$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \tag{2}$$

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \tag{3}$$

$$1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)^{\dagger} \tag{4}$$

$$1.2D \pm 1.0E + 0.5L + 0.2S^{\dagger} \tag{5}$$

$$0.9D \pm (1.6W \text{ or } 1.0E) \tag{6}$$

Combinations 5 and 6 account for the possibility of the dead load and wind or earthquake load counteracting each other; for example, in combination 6, the net load effect could be the difference between $0.9D$ and $1.6W$ or between $0.9D$ and $1.0E$. (Wind or earthquake load may tend to overturn a structure, but the dead load will have a stabilizing effect.)

As previously mentioned, the load factor for a particular load effect is not the same in all load combinations. For example, in combination 2 the load factor for the live load L is 1.6, whereas in combination 3, it is 0.5. The reason is that the live load is being taken as the dominant effect in combination 2, and one of the three effects,

*This load does not include *ponding,* a phenomenon that we discuss in Chapter 5.

†For garages, areas of public assembly, and where the live load exceeds 100 psf, the load factor for the live load L in combinations 3, 4, and 5 should be 1.0 instead of 0.5; that is,

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L + 0.8W) \tag{3}$$

$$1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \tag{4}$$

$$1.2D \pm 1.0E + 1.0L + 0.2S \tag{5}$$

L_r, S, or R, will be dominant in combination 3. In each combination, one of the effects is considered to be at its "lifetime maximum" value and the others at their "arbitrary point in time" values.

The resistance factor ϕ for each type of resistance is given by AISC in the Specification chapter dealing with that resistance, but in most cases, one of two values will be used: 0.90 for limit sates involving yielding or compression buckling and 0.75 for limit states involving rupture (fracture).

2.4 SAFETY FACTORS AND LOAD COMBINATIONS FOR ASD

For allowable strength design, the relationship between loads and strength (Equation 2.1) can be expressed as

$$R_a \leq \frac{R_n}{\Omega} \tag{2.7}$$

where

R_a = required strength
R_n = nominal strength (same as for LRFD)
Ω = safety factor
R_n/Ω = allowable strength

The required strength R_a is the sum of the service loads or load effects. As with LRFD, specific combinations of loads must be considered. Load combinations for ASD are also given in ASCE 7. As with the LRFD combinations, we will omit fluid pressure F, earth pressure H, and self-straining force T. With these omissions, the combinations are

$$D \tag{1}$$

$$D + L \tag{2}$$

$$D + (L_r \text{ or } S \text{ or } R) \tag{3}$$

$$D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) \tag{4}$$

$$D \pm (W \text{ or } 0.7E) \tag{5}$$

$$D + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) \tag{6}$$

$$0.6D \pm (W \text{ or } 0.7E) \tag{7}$$

The factors shown in these combinations are not load factors. The 0.75 factor in some of the combinations accounts for the unlikelihood that all loads in the combination will be at their lifetime maximum values simultaneously. The 0.7 factor applied to the seismic load effect E is used because ASCE 7 uses a strength approach (i.e., LRFD) for computing seismic loads, and the factor is an attempt to equalize the effect for ASD.

Corresponding to the two most common values of resistance factors in LRFD are the following values of the safety factor Ω in ASD: For limit states involving yielding or compression buckling, $\Omega = 1.67$*. For limit states involving rupture, $\Omega = 2.00$. The relationship between resistance factors and safety factors is given by

$$\Omega = \frac{1.5}{\phi} \tag{2.8}$$

For reasons that will be discussed later, this relationship will produce similar designs for LRFD and ASD, under certain loading conditions.

If both sides of Equation 2.7 are divided by area (in the case of axial load) or section modulus (in the case of bending moment), then the relationship becomes

$$f \leq F$$

where
f = applied stress
F = allowable stress

This formulation is called *allowable stress design.*

Example 2.1 A column (compression member) in the upper story of a building is subject to the following loads:

Dead load:	109 kips compression
Floor live load:	46 kips compression
Roof live load:	19 kips compression
Snow:	20 kips compression

a. Determine the controlling load combination for LRFD and the corresponding factored load.
b. If the resistance factor ϕ is 0.90, what is the required *nominal* strength?
c. Determine the controlling load combination for ASD and the corresponding required service load strength.
d. If the safety factor Ω is 1.67, what is the required nominal strength based on the required service load strength?

Solution Even though a load may not be acting directly on a member, it can still cause a load effect in the member. This is true of both snow and roof live load in this example. Although this building is subjected to wind, the resulting forces on the structure are resisted by members other than this particular column.

a. The controlling load combination is the one that produces the largest factored load. We evaluate each expression that involves dead load, D, live load resulting from equipment and occupancy, L, roof live load, L_r, and snow, S.

---

*The value of Ω is actually $1\frac{2}{3} = 5/3$ but has been rounded to 1.67 in the AISC specification.

Combination 1: $1.4D = 1.4(109) = 152.6$ kips

Combination 2: $1.2D + 1.6L + 0.5(L_r$ or S or $R)$. Because S is larger than L_r and $R = 0$, we need to evaluate this combination only once, using S.

$$1.2D + 1.6L + 0.5S = 1.2(109) + 1.6(46) + 0.5(20) = 214.4 \text{ kips}$$

Combination 3: $1.2D + 1.6(L_r$ or S or $R) + (0.5L$ or $0.8W)$. In this combination, we use S instead of L_r, and both R and W are zero.

$$1.2D + 1.6S + 0.5L = 1.2(109) + 1.6(20) + 0.5(46) = 185.8 \text{ kips}$$

Combination 4: $1.2D + 1.6W + 0.5L + 0.5(L_r$ or S or $R)$. This expression reduces to $1.2D + 0.5L + 0.5S$, and by inspection, we can see that it produces a smaller result than combination 3.

Combination 5: $1.2D \pm 1.0E + 0.5L + 0.2S$. As $E = 0$, this expression reduces to $1.2D + 0.5L + 0.2S$, which produces a smaller result than combination 4.

Combination 6: $0.9D \pm (1.6W$ or $1.0E)$. This expression reduces to $0.9D$, which is smaller than any of the other combinations.

Answer Combination 2 controls, and the factored load is 214.4 kips.

b. If the factored load obtained in part (a) is substituted into the fundamental LRFD relationship, Equation 2.6, we obtain

$$R_u \le \phi R_n$$
$$214.4 \le 0.90 R_n$$
$$R_n \ge 238 \text{ kips}$$

Answer The required nominal strength is 238 kips.

c. As with the combinations for LRFD, we will evaluate the expressions involving D, L, L_r, and S for ASD.

Combination 1: $D = 109$ kips. (Obviously this case will never control when live load is present.)

Combination 2: $D + L = 109 + 46 = 155$ kips

Combination 3: $D + (L_r$ or S or $R)$. Since S is larger than L_r, and $R = 0$, this combination reduces to $D + S = 109 + 20 = 129$ kips

Combination 4: $D + 0.75L + 0.75(L_r$ or S or $R)$. This expression reduces to $D + 0.75L + 0.75S = 109 + 0.75(46) + 0.75(20) = 158.5$ kips

Combination 5: $D \pm (W$ or $0.7E)$. Because W and E are zero, this expression reduces to combination 1.

Combination 6: $D + 0.75(W$ or $0.7E) + 0.75L + 0.75\ (L_r$ or S or $R)$. Because W and E are zero, this expression reduces to combination 4.

Combination 7: $0.6D \pm (W$ or $0.7E)$. Because W and E are zero, this expression reduces to $0.6D$, which is smaller than combination 1.

Answer Combination 4 controls, and the required service load strength is 158.5 kips.

d. From the ASD relationship, Equation 2.7,

$$R_a \leq \frac{R_n}{\Omega}$$

$$158.5 \leq \frac{R_n}{1.67}$$

$$R_n \geq 265 \text{ kips}$$

Answer The required nominal strength is 265 kips.

Example 2.1 illustrates that the controlling load combination for LRFD may not control for ASD.

When LRFD was introduced into the AISC Specification in 1986, the load factors were determined in such a way as to give the same results for LRFD and ASD when the loads consisted of dead load and a live load equal to three times the dead load. The resulting relationship between the resistance factor ϕ and the safety factor Ω, as expressed in Equation 2.8, can be derived as follows. Let R_n from Equations 2.6 and 2.7 be the same when $L = 3D$. That is,

$$\frac{R_u}{\phi} = R_a\Omega$$

$$\frac{1.2D + 1.6L}{\phi} = (D + L)\Omega$$

or

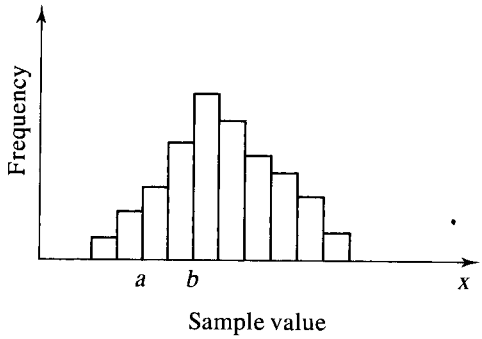
$$\frac{1.2D + 1.6(3D)}{\phi} = (D + 3D)\Omega$$

$$\Omega = \frac{1.5}{\phi}$$

2.5 PROBABILISTIC BASIS OF LOAD AND RESISTANCE FACTORS

Both the load and the resistance factors specified by AISC are based on probabilistic concepts. The resistance factors account for uncertainties in material properties, design theory, and fabrication and construction practices. Although a complete treatment of probability theory is beyond the scope of this book, we present a brief summary of the basic concepts here.

FIGURE 2.1



Experimental data can be represented in the form of a histogram, or bar graph, as shown in Figure 2.1, with the abscissa representing sample values, or events, and the ordinate representing either the number of samples having a certain value or the frequency of occurrence of a certain value. Each bar can represent a single sample value or a range of values. If the ordinate is the percentage of values rather than the actual number of values, the graph is referred to as a *relative* frequency distribution. In such a case the sum of the ordinates will be 100%. If the abscissa values are random events, and enough samples are used, each ordinate can be interpreted as the probability, expressed as a percentage, of that sample value or event occurring. The relative frequency can also be expressed in decimal form, with values between 0 and 1.0. Thus the sum of the ordinates will be unity, and if each bar has a unit width, the total area of the diagram will also be unity. This result implies a probability of 1.0 that an event will fall within the boundaries of the diagram. Furthermore, the probability that a certain value or something smaller will occur is equal to the area of the diagram to the left of that value. The probability of an event having a value falling between a and b in Figure 2.1 equals the area of the diagram between a and b.

Before proceeding, some definitions are in order. The *mean*, $\bar{x}$, of a set of sample values, or *population*, is the arithmetic average, or

$$\bar{x} = \frac{1}{n}\sum_{i=1}^{n} x_i$$

where x_i is a sample value and n is the number of values. The *median* is the middle value of x, and the *mode* is the most frequently occurring value. The *variance*, v, is a measure of the overall variation of the data from the mean and is defined as

$$v = \frac{1}{n}\sum_{i-1}^{n} (x_i - \bar{x})^2$$

The *standard deviation* s is the square root of the variance, or

$$s = \sqrt{\frac{1}{n}\sum_{i=1}^{n} (x_i - \bar{x})^2}$$

Like the variance, the standard deviation is a measure of the overall variation, but it has the same units and the same order of magnitude as the data. The *coefficient of variation, V,* is the standard deviation divided by the mean, or

$$V = \frac{s}{\bar{x}}$$

If the actual frequency distribution is replaced by a theoretical continuous function that closely approximates the data, it is called a *probability density function.* Such a function is illustrated in Figure 2.2. Probability functions are designed so that the total area under the curve is unity. That is, for a function $f(x)$,

$$\int_{-\infty}^{+\infty} f(x)\,dx = 1.0$$

which means that the probability that one of the sample values or events will occur is 1.0. The probability of one of the events between a and b in Figure 2.2 equals the area under the curve between a and b, or

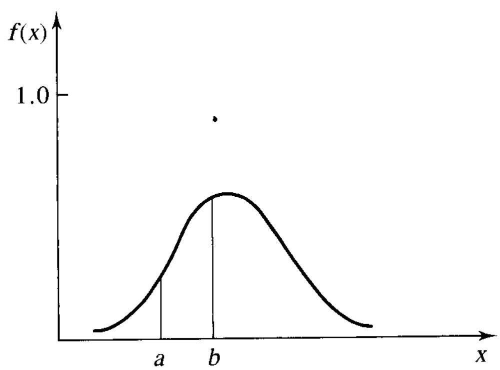
$$\int_{a}^{b} f(x)\,dx$$

When a theoretical probability density function is used, the following notation is conventional:

μ = mean
σ = standard deviation

The probabilistic basis of the load and resistance factors used by AISC is presented in the ASCE structural journal and is summarized here (Ravindra and Galambos, 1978). Load effects, Q, and resistances, R, are random variables and depend on many factors. Loads can be estimated or obtained from measurements and inventories of actual structures, and resistances can be computed or determined experimentally. Discrete values of Q and R from observations can be plotted as frequency distribution histograms or represented by theoretical probability density functions. We use this latter representation in the material that follows.

FIGURE 2.2



If the distributions of Q and R are combined into one function, $R - Q$, positive values of $R - Q$ correspond to survival. Equivalently, if a probability density function of R/Q, the factor of safety, is used, survival is represented by values of R/Q greater than 1.0. The corresponding probability of failure is the probability that R/Q is less than 1; that is,

$$P_F = P\left[\left(\frac{R}{Q}\right) < 1\right]$$

Taking the natural logarithm of both sides of the inequality, we have

$$P_F = P\left[\ln\left(\frac{R}{Q}\right) < \ln 1\right] = P\left[\ln\left(\frac{R}{Q}\right) < 0\right]$$

The frequency distribution curve of $\ln(R/Q)$ is shown in Figure 2.3. The *standardized* form of the variable $\ln(R/Q)$ can be defined as

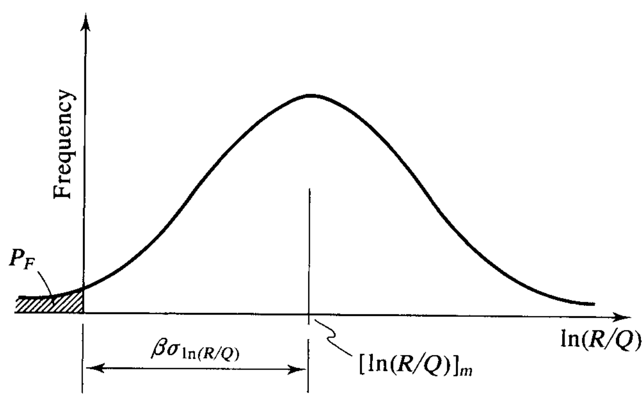
$$U = \frac{\ln\left(\frac{R}{Q}\right) - \left[\ln\left(\frac{R}{Q}\right)\right]_m}{\sigma_{\ln(R/Q)}}$$

where

$$\left[\ln\left(\frac{R}{Q}\right)\right]_m = \text{the mean value of } \ln\left(\frac{R}{Q}\right)$$

$$\sigma_{\ln(R/Q)} = \text{standard deviation of } \ln\left(\frac{R}{Q}\right)$$

FIGURE 2.3



This transformation converts the abscissa U to multiples of standard deviations and places the mean of U at $U = 0$. The probability of failure can then be written as

$$P_F = P\left[\ln\left(\frac{R}{Q}\right) < 0\right] = P\left(\left\{U\sigma_{\ln(R/Q)} + \left[\ln\left(\frac{R}{Q}\right)\right]_m\right\} < 0\right)$$

$$= P\left\{U < -\frac{\left[\ln\left(\frac{R}{Q}\right)\right]_m}{\sigma_{\ln(R/Q)}}\right\} = F_u\left\{-\frac{\left[\ln\left(\frac{R}{Q}\right)\right]_m}{\sigma_{\ln(R/Q)}}\right\}$$

where F_u is the *cumulative distribution function* of U, or the probability that U will not exceed the argument of the function. If we let

$$\beta = \frac{\left[\ln\left(\frac{R}{Q}\right)\right]_m}{\sigma_{\ln(R/Q)}}$$

then

$$\left[\ln\left(\frac{R}{Q}\right)\right]_m = \beta\sigma_{\ln(R/Q)}$$

The variable β can be interpreted as the number of standard deviations from the origin that the mean value of $\ln(R/Q)$ is. For safety, the mean value *must* be more than zero, and as a consequence, β is called the *safety index* or *reliability index*. The larger this value, the larger will be the margin of safety. This means that the probability of failure, represented by the shaded area in Figure 2.3 labeled P_F, will be smaller. The reliability index is a function of both the load effect Q and the resistance R. Use of the same reliability index for all types of members subjected to the same type of loading gives the members relatively uniform strength. The "target" values of β shown in Table 2.1, selected and used in computing both load and resistance factors for the AISC Specification, were based on the recommendations of Ravindra and Galambos (1978), who also showed that

$$\phi = \frac{R_m}{R_n} e^{-0.55\beta V_R}.$$

TABLE 2.1 Target Values of β

Type of Component	Loading Condition: $D + (L \text{ or } S)$	$D + L + W$	$D + L + E$
Members	3.0	2.5	1.75
Connections	4.5	4.5	4.5

where

R_m = mean value of the resistance R
R_n = nominal or theoretical resistance
V_R = coefficient of variation of R

2.6 STEEL CONSTRUCTION MANUAL

Anyone engaged in structural steel design in the United States must have access to AISC's *Steel Construction Manual* (AISC, 2005b). This publication contains the AISC Specification and numerous design aids in the form of tables and graphs, as well as a "catalog" of the most widely available structural shapes.

The first nine editions of the *Manual* and the accompanying specifications were based on ASD. The ninth edition was followed by editions one through three of the LRFD-based manuals. The current version, which incorporates both ASD and LRFD, is therefore the thirteenth edition.

This textbook was written under the assumption that you would have access to the *Manual* at all times. To encourage use of the *Manual,* we did not reproduce its tables and graphs in this book. The *Manual* is divided into 17 parts as follows:

Part 1. Dimensions and Properties. This part contains details on standard hot-rolled shapes, pipe, and hollow structural sections, including all necessary cross-sectional dimensions and properties such as area and moment of inertia.

Part 2. General Design Considerations. This part includes a brief overview of various specifications (including a detailed discussion of the AISC Specification), codes and standards, some fundamental design and fabrication principles, and a discussion of the proper selection of materials.

Part 3. Design of Flexural Members. This part contains a discussion of Specification requirements and design aids for beams, including composite beams (in which a steel shape acts in combination with a reinforced concrete floor or roof slab) and plate girders. Composite beams are covered in Chapter 9 of this textbook, "Composite Construction," and plate girders are covered in Chapter 10, "Plate Girders."

Part 4. Design of Compression Members. Part 4 includes a discussion of the Specification requirements for compression members and numerous design aids. Design aids for composite columns, consisting of hollow structural sections or pipe filled with plain (unreinforced) concrete, are also included. Composite columns are covered in Chapter 9 of this textbook.

Part 5. Design of Tension Members. This part includes design aids for tension members and a summary of the Specification requirements for tension members.

Part 6. Design of Members Subject to Combined Loading. Part 6 covers members subject to combined axial tension and flexure, combined axial compression and flexure, and combined torsion, flexure, shear, and/or axial force. Of particular interest is the material on combined axial compression and flexure, which is the subject of Chapter 6 of this textbook, "Beam–Columns."

Parts 7–15 cover connections:

Part 7. Design Considerations for Bolts.

Part 8. Design Considerations for Welds.

Part 9. Design of Connecting Elements.

Part 10. Design of Simple Shear Connections.

Part 11. Design of Flexible Moment Connections.

Part 12. Design of Fully Restrained (FR) Moment Connections.

Part 13. Design of Bracing Connections and Truss Connections.

Part 14. Design of Beam Bearing Plates, Column Base Plates, Anchor Rods, and Column Splices.

Part 15. Design of Hanger Connections, Bracket Plates, and Crane–Rail Connections.

Part 16. Specifications and Codes. This part contains the AISC Specification and Commentary, a specification for high-strength bolts (RCSC, 2004), and the AISC Code of Standard Practice (AISC, 2005c).

Part 17. Miscellaneous Data and Mathematical Information. This part includes properties of standard steel shapes in SI units, conversion factors and other information on SI units, weights and other properties of building materials, mathematical formulas, and properties of geometric shapes.

All design aids in the *Manual* give values for both allowable strength design (ASD) and load and resistance factor design (LRFD). The *Manual* uses a color-coding scheme for these values: ASD allowable strength values (R_n/Ω) are shown as black numbers on a green background, and LRFD design strength values (ϕR_n) are shown as blue numbers on a white background.

The AISC Specification is only a small part of the *Manual*. Many of the terms and constants used in other parts of the *Manual* are presented to facilitate the design process and are not necessarily part of the Specification. In some instances, the recommendations are only "rules of thumb" based on common practice, not requirements of the Specification. Although such information is not in conflict with the Specification, it is important to recognize what is a *requirement* (when adopted by a building code) and what is not.

The *Manual* is accompanied by a compact disk that contains worked-out examples that illustrate Specification requirements and the use of the design aids in the *Manual*. This companion disk also contains the Specification and Commentary, a database of standard hot-rolled section properties, and web links to other AISC resources.

2.7 DESIGN COMPUTATIONS AND PRECISION

The computations required in engineering design and analysis are done with either a digital computer or an electronic calculator. When doing manual computations with the aid of an electronic calculator, an engineer must make a decision regarding the degree of precision needed. The problem of how many significant figures to use in engineering computations has no simple solution. Recording too many significant digits is misleading and can imply an unrealistic degree of precision. Conversely, recording too few figures can lead to meaningless results. The question of precision was mostly academic before the early 1970s, when the chief calculating tool was the slide rule. The guiding principle at that time was to read and record numbers as accurately as possible, which meant three or four significant figures.

There are many inherent inaccuracies and uncertainties in structural design, including variations in material properties and loads; load estimates sometimes border on educated guesses. It hardly makes sense to perform computations with 12 significant figures and record the answer to that degree of precision when the yield stress is known only to the nearest 10 kips per square inch (two significant figures). Furthermore, data given in the *Steel Construction Manual* has been rounded to three significant figures. To avoid results that are even less precise, however, it is reasonable to assume that the given parameters of a problem, such as the yield stress, are exact and then decide on the degree of precision required in subsequent calculations.

A further complication arises when electronic calculators are used. If all of the computations for a problem are done in one continuous series of operations on a calculator, the number of significant figures used is whatever the calculator uses, perhaps 10 or 12. But if intermediate values are rounded, recorded, and used in subsequent computations, then a consistent number of significant figures will not have been used. Furthermore, the manner in which the computations are grouped will influence the final result. In general, the result will be no more accurate than the least accurate number used in the computation—and sometimes less because of round-off error. For example, consider a number calculated on a 12-digit calculator and recorded to four significant figures. If this number is multiplied by a number expressed to five significant figures, the product will be precise to four significant figures at most, regardless of the number of digits displayed on the calculator. Consequently, it is not reasonable to record this number to more than four significant figures.

It is also unreasonable to record the results of every calculator multiplication or division to a predetermined number of significant figures in order to have a consistent degree of precision throughout. A reasonable approach is to perform operations on the calculator in any convenient manner and record intermediate values to whatever degree of precision is deemed adequate (without clearing the intermediate value from the calculator if it can be used in the next computation). The final results should then be expressed to a precision consistent with this procedure, usually to one significant figure less than the intermediate results, to account for round-off error.

It is difficult to determine what the degree of precision should be for the typical structural steel design problem. Using more than three or four significant figures is probably unrealistic in most cases, and results based on less than three may be too approximate to be of any value. In this book we record intermediate values to three or four digits (usually four), depending on the circumstances, and record final results to three digits. For multiplication and division, each number used in an intermediate calculation should be expressed to four significant figures, and the result should be recorded to four significant figures. For addition and subtraction, determining the location of the right-most significant digit in a column of numbers is done as follows: from the left-most significant digit of all numbers involved, move to the right a number of digits corresponding to the number of significant digits desired. For example, to add 12.34 and 2.234 (both numbers have four significant figures) and round to four significant figures,

$$\begin{array}{r} 12.34 \\ +\ 2.234 \\ \hline 14.574 \end{array}$$

and the result should be recorded as 14.57, even though the fifth digit of the result was significant in the second number. As another example, consider the addition of the following numbers, both accurate to four significant figures:

$$36{,}000 + 1.240 = 36{,}001.24$$

The result should be recorded as 36,000 (four significant figures). When subtracting numbers of almost equal value, significant digits can be lost. For example, in the operation

$$12{,}458.62 - 12{,}462.86 = -4.24$$

four significant figures are lost. To avoid this problem, when subtracting, start with additional significant figures if possible.

When rounding numbers where the first digit to be dropped is a 5 with no digits following, two options are possible. The first is to add 1 to the last digit retained. The other is to use the "odd-add" rule, in which we leave the last digit to be retained unchanged if it is an even number, and add 1 if it is an odd number, making it even. In this book, we follow the first practice. The "odd-add" rule tends to average out the rounding process when many numerical operations are involved, as in statistical methods, but that is not the case in most structural design problems. In addition, most calculators, spreadsheet programs, and other software use the first method, and our results will be consistent with those tools; therefore, we will round up when the first digit dropped is a 5 with no digits following.

Problems

Note All given loads are service loads.

2-1 A column in a building is subjected to the following load effects:

9 kips compression from dead load

5 kips compression from roof live load

6 kips compression from snow

7 kips compression from 3 inches of rain accumulated on the roof

8 kips compression from wind

a. If load and resistance factor design is used, determine the factored load (required strength) to be used in the design of the column. Which AISC load combination controls?

b. What is the required *design* strength of the column?

c. What is the required *nominal* strength of the column for a resistance factor ϕ of 0.90?

d. If allowable strength design is used, determine the required load capacity (required strength) to be used in the design of the column. Which AISC load combination controls?

e. What is the required *nominal* strength of the column for a safety factor Ω of 1.67?

2-2 Repeat Problem 2-1 without the possibility of rain accumulation on the proof.

2-3 A beam is part of the framing system for the floor of an office building. The floor is subjected to both dead loads and live loads. The maximum moment caused by the service dead load is 45 ft-kips, and the maximum moment for the service live load is 63 ft-kips (these moments occur at the same location on the beam and can therefore be combined).

a. If load and resistance factor design is used, determine the maximum factored bending moment (required moment strength). What is the controlling AISC load combination?

b. What is the required *nominal* moment strength for a resistance factor ϕ of 0.90?

c. If allowable strength design is used, determine the required moment strength. What is the controlling AISC load combination?

d. What is the required *nominal* moment strength for a safety factor Ω of 1.67?

2-4 A tension member must be designed for a service dead load of 18 kips and a service live load of 2 kips.

a. If load and resistance factor design is used, determine the maximum factored load (required strength) and the controlling AISC load combination.

b. If allowable strength design is used, determine the maximum load (required strength) and the controlling AISC load combination.

2-5 A flat roof is subject to the following uniformly distributed loads: a dead load of 21 psf (pounds per square foot of roof surface), a roof live load of 12 psf, a snow load of 13.5 psf, and a wind load of 22 psf *upward.* (Although the wind itself is in a horizontal direction, the force that it exerts on this roof is upward. It will be upward regardless of wind direction. The dead, live and snow loads are *gravity loads* and *act downward.*)

a. If load and resistance factor design is used, compute the factored load (required strength) in pounds per square foot. Which AISC load combination controls?

b. If allowable strength design is used, compute the required load capacity (required strength) in pounds per square foot. Which AISC load combination controls?

3 Tension Members

3.1 INTRODUCTION

Tension members are structural elements that are subjected to axial tensile forces. They are used in various types of structures and include truss members, bracing for buildings and bridges, cables in suspended roof systems, and cables in suspension and cable-stayed bridges. Any cross-sectional configuration may be used, because for any given material, the only determinant of the strength of a tension member is the cross-sectional area. Circular rods and rolled angle shapes are frequently used. Built-up shapes, either from plates, rolled shapes, or a combination of plates and rolled shapes, are sometimes used when large loads must be resisted. The most common built-up configuration is probably the double-angle section, shown in Figure 3.1, along with other typical cross sections. Because the use of this section is so widespread, tables of properties of various combinations of angles are included in the AISC *Steel Construction Manual*.

The stress in an axially loaded tension member is given by

$$f = \frac{P}{A}$$

where P is the magnitude of the load and A is the cross-sectional area (the area normal to the load). The stress as given by this equation is exact, provided that the cross section under consideration is not adjacent to the point of application of the load, where the distribution of stress is not uniform.

If the cross-sectional area of a tension member varies along its length, the stress is a function of the particular section under consideration. The presence of holes in a member will influence the stress at a cross section through the hole or holes. At these locations, the cross-sectional area will be reduced by an amount equal to the area removed by the holes. Tension members are frequently connected at their ends with bolts, as illustrated in Figure 3.2. The tension member shown, a $\frac{1}{2} \times 8$ plate, is connected to a *gusset plate*, which is a connection element whose purpose is to transfer the load from the member to a support or to another member. The area of the bar at section *a–a* is $(\frac{1}{2})(8) = 4$ in.^{2}, but the area at section *b–b* is only $4 - (2)(\frac{1}{2})(\frac{7}{8}) = 3.13$ in.^{2} and will be more highly stressed. This reduced area is referred to as the *net area*, or *net section*, and the unreduced area is the *gross area*.

The typical design problem is to select a member with sufficient cross-sectional area to resist the loads. A closely related problem is that of analysis, or review, of a

FIGURE 3.1

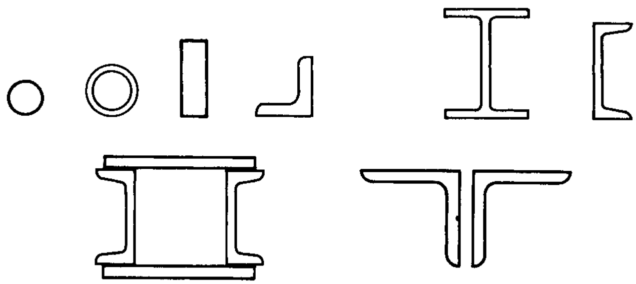
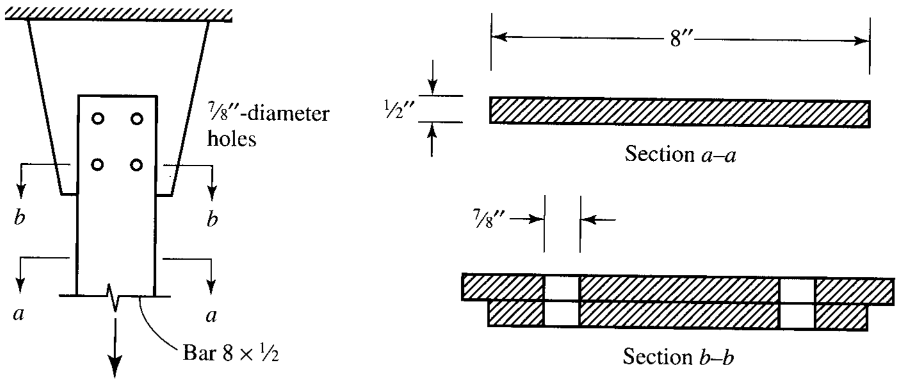


FIGURE 3.2



given member, where in the strength is computed and compared with the load. In general, analysis is a direct procedure, but design is an iterative process and may require some trial and error.

Tension members are covered in Chapter D of the Specification. Requirements that are common with other types of members are covered in Chapter B, "Design Requirements."

3.2 TENSILE STRENGTH

A tension member can fail by reaching one of two limit states: excessive deformation or fracture. To prevent excessive deformation, initiated by yielding, the load on the gross section must be small enough that the stress on the gross section is less than the yield stress F_y. To prevent fracture, the stress on the net section must be less than the tensile strength F_u. In each case, the stress P/A must be less than a limiting stress F or

$$\frac{P}{A} < F$$

Thus the load P must be less than FA, or

$$P < FA$$

The *nominal* strength in yielding is

$$P_n = F_y A_g$$

and the nominal strength in fracture is

$$P_n = F_u A_e$$

where A_e is the *effective* net area, which may be equal to either the net area or, in some cases, a smaller area. We discuss effective net area in Section 3.3.

Although yielding will first occur on the net cross section, the deformation within the length of the connection will generally be smaller than the deformation in the remainder of the tension member. The reason is that the net section exists over a relatively small length of the member, and the total elongation is a product of the length and the strain (a function of the stress). Most of the member will have an unreduced cross section, so attainment of the yield stress on the gross area will result in larger total elongation. It is this larger deformation, not the first yield, that is the limit state.

LRFD: In load and resistance factor design, the factored tensile load is compared to the design strength. The design strength is the resistance factor times the nominal strength. Equation 2.6,

$$R_u = \phi R_n$$

can be written for tension members as

$$P_u \le \phi_t P_n$$

where P_u is the governing combination of factored loads. The resistance factor ϕ_t is smaller for fracture than for yielding, reflecting the more serious nature of fracture.

For yielding, $\phi_t = 0.90$

For fracture, $\phi_t = 0.75$

Because there are two limit states, both of the following conditions must be satisfied:

$$P_u \le 0.90 F_y A_g$$

$$P_u \le 0.75 F_u A_e$$

The smaller of these is the design strength of the member.

ASD: In allowable strength design, the total service load is compared to the allowable strength (allowable load):

$$P_a \le \frac{P_n}{\Omega_t}$$

where P_a is the required strength (applied load), and P_n/Ω_t is the allowable strength. The subscript "a" indicates that the required strength is for "allowable strength design," but you can think of it as standing for "applied" load.

For yielding of the gross section, the safety factor Ω_t is 1.67, and the allowable load is

$$\frac{P_n}{\Omega_t} = \frac{F_y A_g}{1.67} = 0.6 F_y A_g$$

(The factor 0.6 appears to be a rounded value, but recall that 1.67 is a rounded value. If $\Omega_t = 5/3$ is used, the allowable load is exactly 0.6 $F_y A_g$.)

For fracture of the net section, the safety factor is 2.00 and the allowable load is

$$\frac{P_n}{\Omega_t} = \frac{F_u A_e}{2.00} = 0.5 F_u A_e$$

Alternatively, the service load stress can be compared to the allowable stress. This can be expressed as

$$f_t \leq F_t$$

where f_t is the applied stress and F_t is the allowable stress. For yielding of the gross section,

$$f_t = \frac{P_a}{A_g} \quad \text{and} \quad F_t = \frac{P_n/\Omega_t}{A_g} = \frac{0.6 F_y A_g}{A_g} = 0.6 F_y$$

For fracture of the net section,

$$f_t = \frac{P_a}{A_e} \quad \text{and} \quad F_t = \frac{P_n/\Omega_t}{A_e} = \frac{0.5 F_u A_e}{A_e} = 0.5 F_u$$

You can find values of F_y and F_u for various structural steels in Table 2-3 in the *Manual.* All of the steels that are available for various hot-rolled shapes are indicated by shaded areas. The black areas correspond to preferred materials, and the gray areas represent other steels that are available. Under the W heading, we see that A992 is the preferred material for W shapes, but other materials are available, usually at a higher cost. For some steels, there is more than one grade, with each grade having different values of F_y and F_u. In these cases, the grade must be specified along with the ASTM designation—for example, A572 Grade 50. For A242 steel, F_y and F_u depend on the thickness of the flange of the cross-sectional shape. This relationship is given in footnotes in the table. For example, to determine the properties of a W33 $\times$221 of ASTM A242 steel, first refer to the dimensions and properties table in Part 1 of the *Manual* and determine that the flange thickness t_f is equal to 1.28 inches. This matches the thickness range indicated in footnote1; therefore, $F_y = 50$ ksi and $F_u = 70$ ksi. Values of F_y and F_u for plates and bars are given in Table 2-4, and information on structural fasteners, including bolts and rods, can be found in Table 2-5.

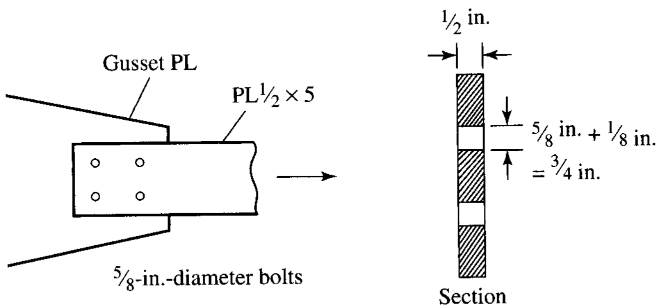
The exact amount of area to be deducted from the gross area to account for the presence of bolt holes depends on the fabrication procedure. The usual practice is to drill or punch standard holes (i.e., not oversized) with a diameter 1/16 inch larger than the fastener diameter. To account for possible roughness around the edges of the hole, Section D3 of the AISC Specification (in the remainder of this book, references to the Specification will usually be in the form AISC D3) requires the addition of 1/16 inch to the actual hole diameter. This amounts to using an effective hole diameter 1/8 inch larger than the fastener diameter. In the case of slotted holes, 1/16 inch should be added to the actual *width* of the hole. You can find details related to standard, oversized, and slotted holes in AISC J3.2, "Size and Use of Holes" (in Chapter J, "Design of Connections").

Example 3.1 A $\frac{1}{2} \times 5$ plate of A36 steel is used as a tension member. It is connected to a gusset plate with four $\frac{5}{8}$-inch-diameter bolts as shown in Figure 3.3. Assume that the effective net area A_e equals the actual net area A_n (we cover computation of effective net area in Section 3.3).

a) What is the design strength for LRFD?

b) What is the allowable strength for ASD?

FIGURE 3.3



Solution For yielding of the gross section,

$$A_g = 5(1/2) = 2.5 \text{ in.}^2$$

and the nominal strength is

$$P_n = F_y A_g = 36(2.5) = 90.0 \text{ kips}$$

For fracture of the net section,

$$A_n = A_g - A_{holes} = 2.5 - (\tfrac{1}{2})(\tfrac{3}{4}) \times 2 \text{ holes}$$
$$= 2.5 - 0.75 = 1.75 \text{ in.}^2$$

$A_e = A_n = 1.75$ in.^{2} (This is true for this example, but A_e does not always equal A_n.)

The nominal strength is

$$P_n = F_u A_e = 58(1.75) = 101.5 \text{ kips}$$

a) The design strength based on yielding is

$$\phi_t P_n = 0.90(90) = 81.0 \text{ kips}$$

The design strength based on fracture is

$$\phi_t P_n = 0.75(101.5) = 76.1 \text{ kips}$$

Answer The design strength for LRFD is the smaller value: $\phi_t P_n = 76.1$ kips.

b) The allowable strength based on yielding is

$$\frac{P_n}{\Omega_t} = \frac{90}{1.67} = 53.9 \text{ kips}$$

The allowable strength based on fracture is

$$\frac{P_n}{\Omega_t} = \frac{101.5}{2.00} = 50.8 \text{ kips}$$

Answer The allowable service load is the smaller value = 50.8 kips.

Alternative Solution Using Allowable Stress: For yielding,

$$F_t = 0.6F_y = 0.6(36) = 21.6 \text{ ksi}$$

and the allowable load is

$$F_tA_g = 21.6(2.5) = 54.0 \text{ kips}$$

(The slight difference between this value and the one based on allowable strength is because the value of Ω in the allowable strength approach has been rounded from 5/3 to 1.67; the value based on the allowable stress is the more accurate one.)
For fracture,

$$F_t = 0.5F_u = 0.5(58) = 29.0 \text{ ksi}$$

and the allowable load is

$$F_tA_e = 29.0(1.75) = 50.8 \text{ kips}$$

Answer The allowable service load is the smaller value = 50.8 kips.

Because of the relationship given by Equation 2.8, the allowable strength will always be equal to the design strength divided by 1.5. In this book, however, we will do the complete computation of allowable strength even when the design strength is available.

The effects of stress concentrations at holes appear to have been overlooked. In reality, stresses at holes can be as high as three times the average stress on the net section, and at fillets of rolled shapes they can be more than twice the average (McGuire, 1968). Because of the ductile nature of structural steel, the usual design practice is to neglect such localized overstress. After yielding begins at a point of stress concentration, additional stress is transferred to adjacent areas of the cross section. This stress redistribution is responsible for the "forgiving" nature of structural steel. Its ductility permits the initially yielded zone to deform without fracture as the stress on the remainder of the cross section continues to increase. Under certain conditions, however, steel may lose its ductility and stress concentrations can precipitate brittle fracture. These situations include fatigue loading and extremely low temperature.

Example 3.2 A single-angle tension member, an L3½ × 3½ × ⅜, is connected to a gusset plate with ⅞-inch-diameter bolts as shown in Figure 3.4. A36 steel is used. The service loads are 35 kips dead load and 15 kips live load. Investigate this member for compliance with the AISC Specification. Assume that the effective net area is 85% of the computed net area.

a) Use LRFD.

b) Use ASD.