Preface

This textbook has been prepared with the hope that its readers will, as have so many engineers in the past, become interested in structural steel design and want to maintain and increase their knowledge on the subject throughout their careers in the engineering and construction industries. The material was prepared primarily for an introductory course in the junior or senior year but the last several chapters may be used for a graduate class. The authors have assumed that the student has previously taken introductory courses in mechanics of materials and structural analysis.

The authors' major objective in preparing this new edition was to update the text to conform to both the American Institute of Steel Construction (AISC) 2016 Specification for Structural Steel Buildings (ANSI/AISC 360-16) and the 15th edition of the AISC Steel Construction Manual published in 2017.

WHAT'S NEW IN THIS EDITION

Several changes to the text were made to the textbook in this edition:

1. Most of the end of chapter *Problems for Solution* for Chapters 2 through 11 have been revised. For Chapters 12 through 18 about half the problems have been revised.
2. For Chapters 2 through 14, Examples with Video Solutions have been added.
3. The load factors and load combinations defined in Chapter 2 of the textbook and used throughout the book in example problems and end of chapter problems for solution have been revised to meet those given in the ASCE 7-16.
4. The classification of compression sections defined in Chapters 5, 6 and 7 of the textbook has been revised to the new definition of Table 4-1 in the 15th edition AISC Steel Manual. The available axial compression strengths are given for W-shaped members in Table 4-1a for $F_y = 50$ ksi, Table 4-1b for $F_y = 65$ ksi, Table 4-1c for $F_y = 70$ ksi steel sections.
5. A change has been made in the preferred material specification for round and rectangular HSS section. The ASTM Designation is now A500, Grade C with $F_y = 50$ ksi for rectangular sections and $F_y = 46$ ksi for round sections.
6. A change has been made based on AISC adopting an ASTM umbrella bolt specification, ASTM F3125, that includes Grades A325, A325M, A490, A490M, F1852 and F2280.
7. In Chapter 3 of the textbook, the Case 4 Shear Lag factor shown in Table 3.2 was revised for welded plates or connected elements with unequal length longitudinal welds.

8. In Chapter 6 and Appendix C of the textbook, changes for determining the available compressive strength for a variety section types.
9. In Chapter 11 of the textbook, revisions were made to incorporate a change in Part 6 of the 15th edition AISC Steel Manual. Table 6-1 "Combined Flexure and Axial Force" was replaced with Table 6-2 "Strengths and Properties for Axial, Flexural and Combined Forces".
10. In Chapter 18 of the textbook, revisions were made for the shear strength of webs with or without tension field action.
11. Nomenclature changes were made throughout the textbook based on changes in the AISC Specification ANSI/AISC 360-16.
12. Various photos were updated throughout the textbook.

The authors would like to express appreciation to Michael Stoner a Clemson University graduate student, for his assistance in the review of the end of chapter problems and their solutions. In addition, the American Institute of Steel Construction was very helpful in providing the advance copies of the AISC Specification and Steel Construction Manual revisions. Finally, we would like to thank our families for their encouragement and support in the revising of the manuscript of this textbook.

We also thank the reviewers and users of the previous editions of this book for their suggestions, corrections, and criticisms. We welcome any comments on this edition.

Jack C. McCormac, P.E.
Stephen F. Csernak, P.E.

Contents

CHAPTER 1

Introduction to Structural Steel Design

1.1 ADVANTAGES OF STEEL AS A STRUCTURAL MATERIAL

A person traveling in the United States might quite understandably decide that steel is the perfect structural material. He or she would see an endless number of steel bridges, buildings, towers, and other structures. After seeing these numerous steel structures, this traveler might be surprised to learn that steel was not economically made in the United States until late in the nineteenth century, and the first wide-flange beams were not rolled until 1908.

The assumption of the perfection of this metal, perhaps the most versatile of structural materials, would appear to be even more reasonable when its great strength, light weight, ease of fabrication, and many other desirable properties are considered. These and other advantages of structural steel are discussed in detail in the paragraphs that follow.

1.1.1 High Strength

The high strength of steel per unit of weight means that the weight of structures will be small. This fact is of great importance for long-span bridges, tall buildings, and structures situated on poor foundations.

1.1.2 Uniformity

The properties of steel do not change appreciably with time, as do those of a reinforced-concrete structure.

1.1.3 Elasticity

Steel behaves closer to design assumptions than most materials because it follows Hooke's law up to fairly high stresses. The moments of inertia of a steel structure can be accurately calculated, while the values obtained for a reinforced-concrete structure are rather indefinite.

Erection of steel joists. (Courtesy of Nucor Vulcraft Verco Group.)

1.1.4 Permanence

Steel frames that are properly maintained will last indefinitely. Research on some of the newer steels indicates that under certain conditions no painting maintenance whatsoever will be required.

1.1.5 Ductility

The property of a material by which it can withstand extensive deformation without failure under high tensile stresses is its *ductility*. When a *mild-* or *low-carbon* structural steel member is being tested in tension, a considerable reduction in cross section and a large amount of elongation will occur at the point of failure before the actual fracture occurs. A material that does not have this property is generally unacceptable and is probably hard and brittle, and it might break if subjected to a sudden shock.

In structural members under normal loads, high stress concentrations develop at various points. The ductile nature of the usual structural steels enables them to yield locally at those points, thus preventing premature failures. A further advantage of ductile structures is that when overloaded, their large deflections give visible evidence of impending failure (sometimes jokingly referred to as "running time").

1.1.6 Toughness

Structural steels are tough—that is, they have both strength and ductility. A steel member loaded until it has large deformations will still be able to withstand large forces. This is a very important characteristic, because it means that steel members can be subjected to large deformations during fabrication and erection without fracture—thus allowing them to be bent, hammered, and sheared, and to have holes punched in them without visible damage. The ability of a material to absorb energy in large amounts is called *toughness*.

1.1.7 Additions to Existing Structures

Steel structures are quite well suited to having additions made to them. New bays or even entire new wings can be added to existing steel frame buildings, and steel bridges may often be widened.

1.1.8 Miscellaneous

Several other important advantages of structural steel are as follows: (a) ability to be fastened together by several simple connection devices, including welds and bolts; (b) adaptation to prefabrication; (c) speed of erection; (d) ability to be rolled into a wide variety of sizes and shapes, as described in Section 1.4 of this chapter; (e) possible reuse after a structure is disassembled; and (f) scrap value, even though not reusable in its existing form. Steel is the ultimate recyclable material.

1.2 DISADVANTAGES OF STEEL AS A STRUCTURAL MATERIAL

In general, steel has the following disadvantages:

1.2.1 Corrosion

Most steels are susceptible to corrosion when freely exposed to air and water, and therefore must be painted periodically. The use of weathering steels, however, in suitable applications tends to eliminate this cost.

Though weathering steels can be quite effective in certain situations for limiting corrosion, there are many cases where their use is not feasible. In some of these situations, corrosion may be a real problem. For instance, corrosion-fatigue failures can occur where steel members are subject to cyclic stresses and corrosive environments. The fatigue strength of steel members can be appreciably reduced when the members are used in aggressive chemical environments and subject to cyclic loads.

The reader should note that steels are available in which copper is used as an anticorrosion component. The copper is usually absorbed during the steelmaking process.

1.2.2 Fireproofing Costs

Although structural members are incombustible, their strength is tremendously reduced at temperatures commonly reached in fires when the other materials in a building burn. Many disastrous fires have occurred in empty buildings where the only fuel for the fires was the buildings themselves. Furthermore, steel is an excellent heat conductor—nonfireproofed steel members may transmit enough heat from a burning section or compartment of a building to ignite materials with which they are in contact in adjoining sections of the building. As a result, the steel frame of a building may have to be protected by materials with certain insulating characteristics, and the building may have to include a sprinkler system if it is to meet the building code requirements of the locality in question.

1.2.3 Susceptibility to Buckling

As the length and slenderness of a compression member is increased, its danger of buckling increases. For most structures, the use of steel columns is very economical because of their high strength-to-weight ratios. Occasionally, however, some additional steel is needed to stiffen them so they will not buckle. This tends to reduce their economy.

1.2.4 Fatigue

Another undesirable property of steel is that its strength may be reduced if it is subjected to a large number of stress reversals or even to a large number of variations of tensile stress. (Fatigue problems occur only when tension is involved.) The present practice is to reduce the estimations of strength of such members if it is anticipated that they will have more than a prescribed number of cycles of stress variation.

1.2.5 Brittle Fracture

Under certain conditions steel may lose its ductility, and brittle fracture may occur at places of stress concentration. Fatigue-type loadings and very low temperatures aggravate the situation. Triaxial stress conditions can also lead to brittle fracture.

1.3 EARLY USES OF IRON AND STEEL

Although the first metal used by human beings was probably some type of copper alloy such as bronze (made with copper, tin, and perhaps some other additives), the most important metal developments throughout history have occurred in the manufacture and use of iron and its famous alloy called steel. Today, iron and steel make up nearly 95 percent of all the tonnage of metal produced in the world.[1]

Despite diligent efforts for many decades, archaeologists have been unable to discover when iron was first used. They did find an iron dagger and an iron bracelet in the Great Pyramid in Egypt, which they claim had been there undisturbed for at least 5000 years. The use of iron has had a great influence on the course of civilization since the earliest times and may very well continue to do so in the centuries ahead. Since the beginning of the Iron Age in about 1000 BC, the progress of civilization in peace and war has been heavily dependent on what people have been able to make with iron. On many occasions its use has decidedly affected the outcome of military engagements. For instance, in 490 BC in Greece at the Battle of Marathon, the greatly outnumbered Athenians killed 6400 Persians and lost only 192 of their own men. Each of the victors wore 57 pounds of iron armor in the battle. (This was the battle from which the runner Pheidippides ran the approximately 25 miles to Athens and died while shouting news of the victory.) This victory supposedly saved Greek civilization for many years.

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[1]American Iron and Steel Institute, *The Making of Steel* (Washington, DC, not dated), p. 6.

Littlejohn Coliseum—Roof Replacement, Clemson, SC. (Courtesy of CMC South Carolina Steel.)

According to the classic theory concerning the first production of iron in the world, there was once a great forest fire on Mount Ida in Ancient Troy (now Turkey) near the Aegean Sea. The land surface reportedly had a rich content of iron, and the heat of the fire is said to have produced a rather crude form of iron that could be hammered into various shapes. Many historians believe, however, that human beings first learned to use iron which fell to the earth in the form of meteorites. Frequently, the iron in meteorites is combined with nickel to produce a harder metal. Perhaps, early human beings were able to hammer and chip this material into crude tools and weapons.

Steel is defined as a combination of iron and a small amount of carbon, usually less than 1 percent. It also contains small percentages of some other elements. Although some steel has been made for at least 2000–3000 years, there was really no economical production method available until the middle of the nineteenth century.

The first steel almost certainly was obtained when the other elements necessary for producing it were accidentally present when iron was heated. As the years went by, steel probably was made by heating iron in contact with charcoal. The surface of the iron absorbed some carbon from the charcoal, which was then hammered into the hot iron. Repeating this process several times resulted in a case-hardened exterior of steel. In this way the famous swords of Toledo and Damascus were produced.

The first large volume process for producing steel was named after Sir Henry Bessemer of England. He received an English patent for his process in 1855, but his efforts to obtain a U.S. patent for the process in 1856 were unsuccessful, because it was shown that William Kelly of Eddyville, Kentucky, had made steel by the same process seven years before Bessemer applied for his English patent. Although Kelly was given the patent, the name Bessemer was used for the process.[2]

Kelly and Bessemer learned that a blast of air through molten iron burned out most of the impurities in the metal. Unfortunately, at the same time, the blow eliminated some desirable elements such as carbon and manganese. It was later learned that these needed elements could be restored by adding spiegeleisen, which is an alloy of iron, carbon, and manganese. It was further learned that the addition of limestone in the converter resulted in the removal of the phosphorus and most of the sulfur.

Before the Bessemer process was developed, steel was an expensive alloy used primarily for making knives, forks, spoons, and certain types of cutting tools. The Bessemer process reduced production costs by at least 80 percent and allowed, for the first time, production of large quantities of steel.

The Bessemer converter was commonly used in the United States until the beginning of the twentieth century, but since that time it has been replaced with better methods, such as the open-hearth process and the basic oxygen process.

As a result of the Bessemer process, structural carbon steel could be produced in quantity by 1870, and by 1890, steel had become the principal structural metal used in the United States.

Today, most of the structural steel shapes and plates produced in the United States are made by melting scrap steel. This scrap steel is obtained from junk cars and scrapped structural shapes, as well as from discarded refrigerators, motors, typewriters, bed springs, and other similar items. The molten steel is poured into molds that have approximately the final shapes of the members. The resulting sections, which are run through a series of rollers to squeeze them into their final shapes, have better surfaces and fewer internal or residual stresses than newly made steel.

The shapes may be further processed by cold rolling, by applying various coatings, and perhaps by the process of *annealing*. This is the process by which the steel is heated to an intermediate temperature range (say, 1300–1400°F), held at that temperature for several hours, and then allowed to slowly cool to room temperature. Annealing results in steel with less hardness and brittleness, but greater ductility.

The term **wrought iron** refers to iron with a very low carbon content ($\leq$ 0.15 percent), while iron with a very high carbon content ($\geq$ 2 percent) is referred to as **cast iron**. Steel falls in between cast iron and wrought iron and has carbon contents in the range of 0.15 percent to 1.7 percent (as described in Section 1.8 of this chapter).

The first use of metal for a sizable structure occurred in England in Shropshire (about 140 miles northwest of London) in 1779, when cast iron was used for the construction of the 100-ft Coalbrookdale Arch Bridge over the River Severn. It is said that this bridge (which still stands) was a turning point in engineering history because it changed the course of the Industrial Revolution by introducing iron as a structural material. This iron was supposedly four times as strong as stone and thirty times as strong as wood.[3]

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[2]American Iron and Steel Institute, *Steel 76* (Washington, DC, 1976), pp. 5–11.

[3]M. H. Sawyer, "World's First Iron Bridge," *Civil Engineering* (New York: ASCE, December 1979), pp. 46–49.

A number of other cast-iron bridges were constructed in the following decades, but soon after 1840 the more malleable wrought iron began to replace cast iron. The development of the Bessemer process and subsequent advances, such as the open-hearth process, permitted the manufacture of steel at competitive prices. This encouraged the beginning of the almost unbelievable developments of the last 160 years with structural steel.

1.4 STEEL SECTIONS

The first structural shapes made in the United States were angle irons rolled in 1819. I-shaped steel sections were first rolled in the United States in 1884, and the first skeleton frame structure (the Home Insurance Company Building in Chicago) was erected that same year. Credit for inventing the "skyscraper" is usually given to engineer William LeBaron Jenny, who planned the building, apparently during a bricklayers' strike. Prior to this time, tall buildings in the United States were constructed with load-bearing brick walls that were several feet thick.

For the exterior walls of the 10-story building, Jenny used cast-iron columns encased in brick. The beams for the lower six floors were made from wrought iron, while structural steel beams were used for the upper floors. The first building completely framed with structural steel was the second Rand-McNally building, completed in Chicago in 1890.

An important feature of the 985-ft wrought-iron Eiffel tower constructed in 1889 was the use of mechanically operated passenger elevators. The availability of these machines, along with Jenny's skeleton frame idea, led to the construction of thousands of high-rise buildings throughout the world during the last century.

During these early years, the various mills rolled their own individual shapes and published catalogs providing the dimensions, weight, and other properties of the shapes. In 1896, the Association of American Steel Manufacturers (now the American Iron and Steel Institute, or AISI) made the first efforts to standardize shapes. Today, nearly all structural shapes are standardized, though their exact dimensions may vary just a little from mill to mill.[4]

Structural steel can be economically rolled into a wide variety of shapes and sizes without appreciably changing its physical properties. Usually, the most desirable members are those with large moments of inertia in proportion to their areas. The **I**, **T**, and **C** shapes, so commonly used, fall into this class.

Steel sections are usually designated by the shapes of their cross sections. As examples, there are angles, tees, zees, and plates. It is necessary, however, to make a definite distinction between American standard beams (called *S beams*) and wide-flange beams (called *W beams*), as both of them are I-shaped. The inner surface of the flange of a W section is either parallel to the outer surface or nearly so, with a maximum slope of 1 to 20 on the inner surface, depending on the manufacturer.

The S beams, which were the first beam sections rolled in America, have a slope on their inside flange surfaces of 1 to 6. It might be noted that the constant (or nearly

[4]W. McGuire, *Steel Structures* (Englewood Cliffs, NJ: Prentice-Hall, 1968), pp. 19–21.

Pedestrian bridge for North Carolina Cancer Hospital, Chapel Hill, NC. (Courtesy of CMC South Carolina Steel.)

constant) thickness of W-beam flanges compared with the tapered S-beam flanges may facilitate connections. Wide-flange beams comprise nearly 50 percent of the tonnage of structural steel shapes rolled today. The W and S sections are shown in Fig. 1.1, together with several other familiar steel sections. The uses of these various shapes will be discussed in detail in the chapters to follow.

Constant reference is made throughout this book to the 15th edition of the **Steel Construction Manual**, published by the American Institute of Steel Construction (AISC). This manual, which provides detailed information for structural steel shapes, is referred to hereafter as "the AISC Manual," "the Steel Manual," or simply, "the Manual." It is based on the 2016 **Specification for Structural Steel Buildings** (ANSI/AISC 360-16) (hereafter, "the AISC Specification"), published by the AISC on July 07, 2016.

Structural shapes are identified by a certain system described in the Manual for use in drawings, specifications, and designs. This system is standardized so that all steel mills can use the same identification for purposes of ordering, billing, etc. In addition, so much work is handled today with computers and other automated equipment that it is necessary to have a letter-and-number system which can be printed out with a standard keyboard (as opposed to the old system where certain symbols were used for angles, channels, etc.). Examples of this identification system are as follows:

1. A W27 × 114 is a W section approximately 27 in deep, weighing 114 lb/ft.
2. An S12 × 35 is an S section 12 in deep, weighing 35 lb/ft.

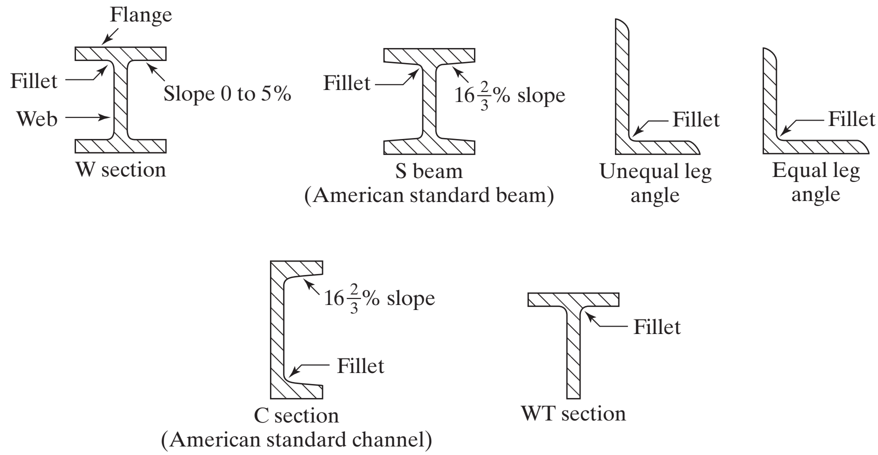


FIGURE 1.1

Rolled-steel shapes.

3. An HP12 × 74 is a bearing pile section approximately 12 in deep, weighing 74 lb/ft. Bearing piles are made with the regular W rolls, but with thicker webs to provide better resistance to the impact of pile driving. The width and depth of these sections are approximately equal, and the flanges and webs have equal or almost equal thickness.
4. An M8 × 6.5 is a miscellaneous section 8 in deep, weighing 6.5 lb/ft. It is one of a group of doubly symmetrical H-shaped members that cannot by dimensions be classified as a W, S, or HP section, as the slope of their inner flanges is other than 16 2/3 percent.
5. A C10 × 30 is a channel 10 in deep, weighing 30 lb/ft.
6. An MC18 × 58 is a miscellaneous channel 18 in deep, weighing 58 lb/ft, which cannot be classified as a C shape because of its dimensions.
7. An HSS14 × 10 × 5/8 is a rectangular hollow structural section 14 in deep, 10 in wide, with a 5/8-in wall thickness. It weighs 93.34 lb/ft. Square and round HSS sections are also available.
8. An L6 × 6 × 1/2 is an equal leg angle, each leg being 6 in long and 1/2 in thick.
9. A WT18 × 151 is a tee obtained by splitting a W36 × 302. This type of section is known as a structural tee.
10. Rectangular steel sections are classified as wide *plates* or narrow *bars*.

The only differences between bars and plates are their sizes and production procedures. Historically, flat pieces have been called bars if they are 8 in or less in width. They are plates if wider than 8 in. Tables 1-29, 2-3, and 2-5 in the AISC Manual provide information on bars and plates. Bar and plate thicknesses are usually specified to the nearest 1/16 in for thicknesses less than 3/8 in, to the nearest 1/8 in for thicknesses between 3/8 in and 1 in, and to the nearest 1/4 in for thicknesses greater than 1 in. A plate is usually designated by its thickness, width, and length, in that order; for example,

Roof framing for Glen Oaks School, Bellerose, NY. (Courtesy of CMC South Carolina Steel.)

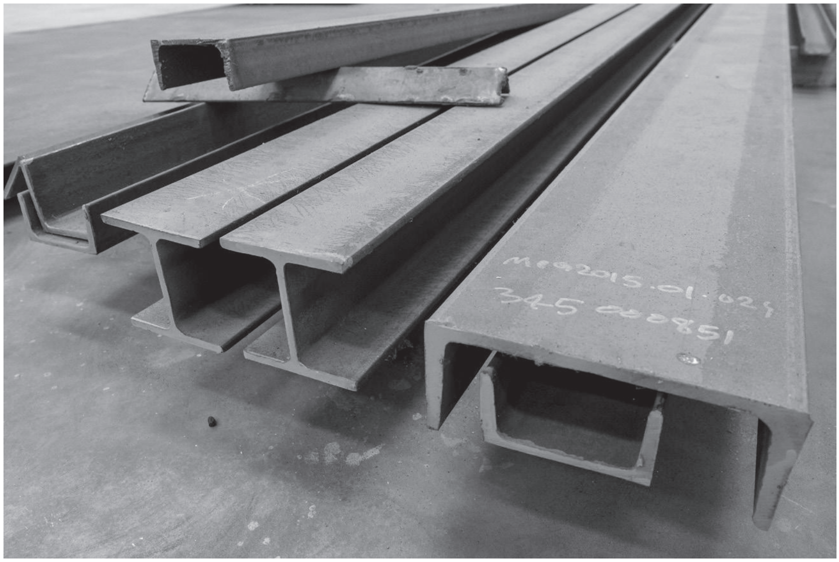
a PL1/2 × 10 × 1 ft 4 in is 1/2 in thick, 10 in wide, and 16 in long. Actually, the term **plate** is almost universally used today, whether a member is fabricated from plate or bar stock. Sheet and strip are usually thinner than bars and plates.

The student should refer to the Steel Manual for information concerning other shapes. Detailed information on these and other sections will be presented herein as needed.

In Part 1 of the Manual, the dimensions and properties of W, S, C, and other shapes are tabulated. The dimensions of the members are given in decimal form (for the use of designers) and in fractions to the nearest sixteenth of an inch (for the use of craftsmen and steel detailers or drafters). Also provided for the use of designers are items such as moments of inertia, section moduli, radii of gyration, and other cross-sectional properties discussed later in this text.

There are variations present in any manufacturing process, and the steel industry is certainly no exception. As a result, the cross-sectional dimensions of steel members may vary somewhat from the values specified in the Manual. Maximum tolerances for the rolling of steel shapes are prescribed by the American Society for Testing and Materials (ASTM) A6 Specification and are presented in Tables 1-22 to 1-28 in the Manual. As a result, calculations can be made on the basis of the properties given in the Manual, regardless of the manufacturer.

Some steel sections listed in the Manual are available in the United States from only one or two steel producers and thus, on occasion, may be difficult to obtain promptly. Accordingly, when specifying sections, the designer would be wise to contact a steel fabricator for a list of sections readily available.



Typical hot-rolled structural steel sections. (Pruch Kijkla/123RF.)

Through the years, there have been changes in the sizes of steel sections. For instance, there may be insufficient demand to continue rolling a certain shape; an existing shape may be dropped because a similar size, but more efficient, shape has been developed, and so forth. Occasionally, designers may need to know the properties of one of the discontinued shapes no longer listed in the latest edition of the Manual or in other tables normally available to them.

For example, it may be desired to add another floor to an existing building that was constructed with shapes no longer rolled. In 1953, the AISC published a book entitled *Iron and Steel Beams 1873 to 1952*, which provides a complete listing of iron and steel beams and their properties rolled in the United States during that period. An up-to-date edition of this book is now available. It is *AISC Design Guide 15* and covers properties of steel shapes produced from 1887 to 2000.[5] There will undoubtedly be many more shape changes in the future. For this reason, the wise structural designer should carefully preserve old editions of the Manual so as to have them available when the older information is needed.

1.5 METRIC UNITS

Almost all of the examples and homework problems presented in this book make use of U.S. customary units. The author, however, feels that today's designer must be able to perform his or her work in either customary or metric units.

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[5]R. L. Brockenbrough, *AISC Rehabilitation and Retrofit Guide: A Reference for Historic Shapes and Specifications* (Chicago, AISC, 2002).

The problem of working with metric units when performing structural steel design in the United States has almost been eliminated by the AISC. Almost all of their equations are written in a form applicable to both systems. In addition, the metric equivalents of the standard U.S. shapes are provided in Section 17 of the Manual. For instance, a W36 × 302 section is shown there as a W920 × 449, where the 920 is mm and the 449 is kg/m.

1.6 COLD-FORMED LIGHT-GAGE STEEL SHAPES

In addition to the hot-rolled steel shapes discussed in the previous section, there are some cold-formed steel shapes. These are made by bending thin sheets of carbon or low-alloy steels into almost any desired cross section, such as the ones shown in Fig. 1.2.[6] These shapes—which may be used for light members in roofs, floors, and walls—vary in thickness from about 0.01 in up to about 0.25 in. The thinner shapes are most often used for some structural panels.

Though cold working does reduce ductility somewhat, it causes some strength increases. Under certain conditions, design specifications will permit the use of these higher strengths.

Concrete floor slabs are very often cast on formed steel decks that serve as economical forms for the wet concrete and are left in place after the concrete hardens. Several types of decking are available, some of which are shown in Fig. 1.3. The sections with the deeper cells have the useful feature that electrical and mechanical conduits can be placed in them. The use of steel decks for floor slabs is discussed in Chapter 16 of this book. There, composite construction is presented. With such construction, steel beams are made composite with concrete slabs by providing for shear transfer between the two so that they will act together as a unit.

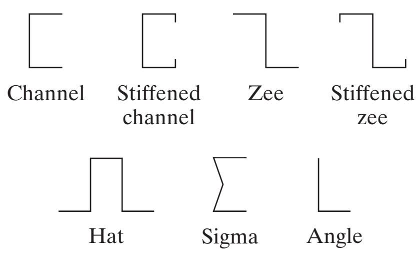

FIGURE 1.2

Cold-formed shapes.

[6]*Cold-formed Steel Design Manual* (Washington, DC: American Iron and Steel Institute, 2013).

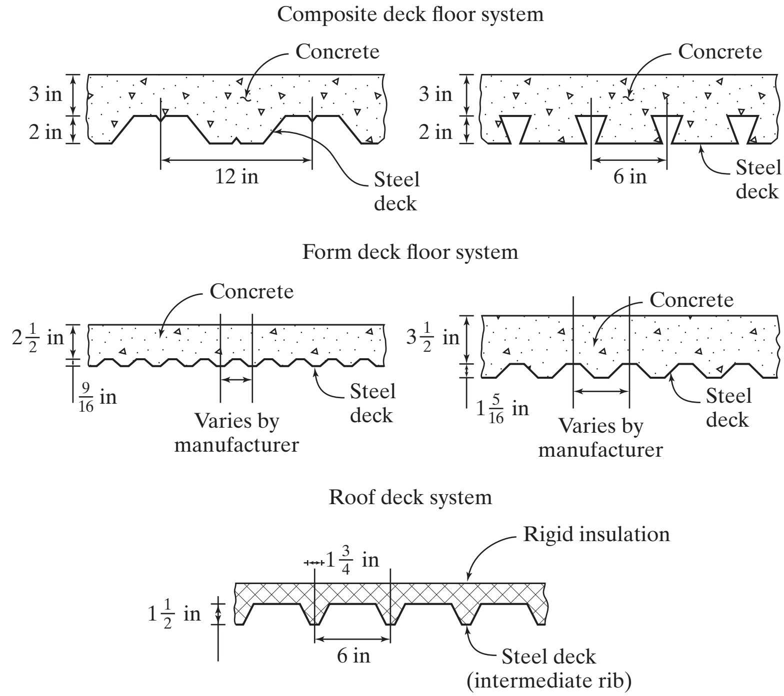

FIGURE 1.3

Some types of steel decks.

1.7 STRESS–STRAIN RELATIONSHIPS IN STRUCTURAL STEEL

To understand the behavior of steel structures, an engineer must be familiar with the properties of steel. Stress–strain diagrams present valuable information necessary to understand how steel will behave in a given situation. Satisfactory steel design methods cannot be developed unless complete information is available concerning the stress–strain relationships of the material being used.

If a piece of ductile structural steel is subjected to a tensile force, it will begin to elongate. If the tensile force is increased at a constant rate, the amount of elongation will increase linearly within certain limits. In other words, elongation will double when the stress goes from 6000 to 12,000 psi (pounds per square inch). When the tensile stress reaches a value roughly equal to three-fourths of the ultimate strength of the steel, the elongation will begin to increase at a greater rate without a corresponding increase in the stress.

The largest stress for which Hooke's law applies, or the highest point on the linear portion of the stress–strain diagram, is called the *proportional limit*. The largest stress that a material can withstand without being permanently deformed is called the *elastic limit*. This value is seldom actually measured and for most engineering materials, including structural steel, is synonymous with the proportional limit. For this reason, the term *proportional elastic limit* is sometimes used.

The stress at which there is a significant increase in the elongation, or strain, without a corresponding increase in stress is said to be the *yield* stress. It is the first point on the stress–strain diagram where a tangent to the curve is horizontal. The yield stress is probably the most important property of steel to the designer, as so many design procedures are based on this value. Beyond the yield stress there is a range in which a considerable increase in strain occurs without increase in stress. The strain that occurs before the yield stress is referred to as the *elastic strain*; the strain that occurs after the yield stress, with no increase in stress, is referred to as the *plastic strain*. Plastic strains are usually from 10 to 15 times as large as the elastic strains.

Yielding of steel without stress increase may be thought to be a severe disadvantage, when in actuality it is a very useful characteristic. It has often performed the wonderful service of preventing failure due to omissions or mistakes on the designer's part. Should the stress at one point in a ductile steel structure reach the yield point, that part of the structure will yield locally without stress increase, thus preventing premature failure. This ductility allows the stresses in a steel structure to be redistributed. Another way of describing this phenomenon is to say that very high stresses caused by fabrication, erection, or loading will tend to equalize themselves. It might also be said that a steel structure has a reserve of plastic strain that enables it to resist overloads and sudden shocks. If it did not have this ability, it might suddenly fracture, like glass or other vitreous substances.

Erection of roof truss, North Charleston, SC. (Courtesy of CMC South Carolina Steel.)

Following the plastic strain, there is a range in which additional stress is necessary to produce additional strain. This is called *strain hardening*. This portion of the diagram is not too important to today's designer, because the strains are so large. A familiar stress–strain diagram for mild- or low-carbon structural steel is shown in Fig. 1.4. Only the initial part of the curve is shown here because of the great deformation that occurs before failure. At failure in the mild steels, the total strains are from 150 to 200 times the elastic strains. The curve will actually continue up to its maximum stress value and then "tail off" before failure. A sharp reduction in the cross section of the member (called *necking*) takes place just before the member fractures.

The stress–strain curve of Fig. 1.4 is typical of the usual ductile structural steel and is assumed to be the same for members in tension or compression. (The compression members must be stocky, because slender compression members subjected to compression loads tend to buckle laterally, and their properties are greatly affected by the bending moments so produced.) The shape of the diagram varies with the speed of loading, the type of steel, and the temperature. One such variation is shown in the figure by the dotted line marked *upper yield*.

This shape stress–strain curve is the result when a mild steel has the load applied rapidly, while the *lower yield* is the case for slow loading.

Figure 1.5 shows typical stress–strain curves for several different yield stress steels.

You should note that the stress–strain diagrams of Figs. 1.4 and 1.5 were prepared for a mild steel at room temperature. During welding operations and during fires, structural steel members may be subjected to very high temperatures.

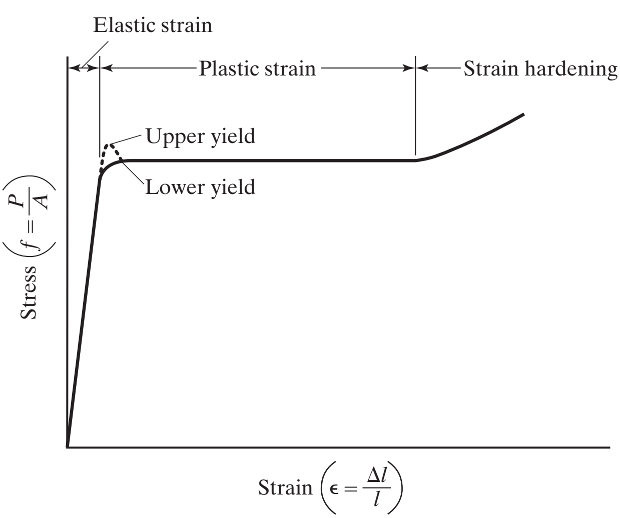

FIGURE 1.4

Typical stress–strain diagram for a mild- or low-carbon structural steel at room temperature.

Steel framed commercial building with single diagonal lateral bracing.
(stocksolutions/123RF.)

Stress–strain diagrams prepared for steels with temperatures above 200°F will be more rounded and nonlinear and will not exhibit well-defined yield points. Steels (particularly those with high carbon contents) may actually increase a little in tensile strength as they are heated to a temperature of about 700°F. As temperatures are raised upto 800°F-to-1000°F range, strengths are drastically reduced, and at 1200°F little strength is left.

Figure 1.6 shows the variation of yield strengths for several grades of steel as their temperatures are raised from room temperature up to 1800°F to 1900°F. Temperatures of the magnitudes shown can easily be reached in steel members during fires, in localized areas of members when welding is being performed, in members in foundries over open flame, and so on.

When steel sections are cooled below 32°F, their strengths will increase a little, but they will have substantial reductions in ductility and toughness.

A very important property of a structure that has been stressed, but not beyond its yield point, is that it will return to its original length when the loads are removed. Should it be stressed beyond this point, it will return only part of the way back to its original position. This knowledge leads to the possibility of testing an existing structure by loading and unloading. If, after the loads are removed, the structure does not resume its original dimensions, it has been stressed beyond its yield point.

Steel is an alloy consisting almost entirely of iron (usually over 98 percent). It also contains small quantities of carbon, silicon, manganese, sulfur, phosphorus, and

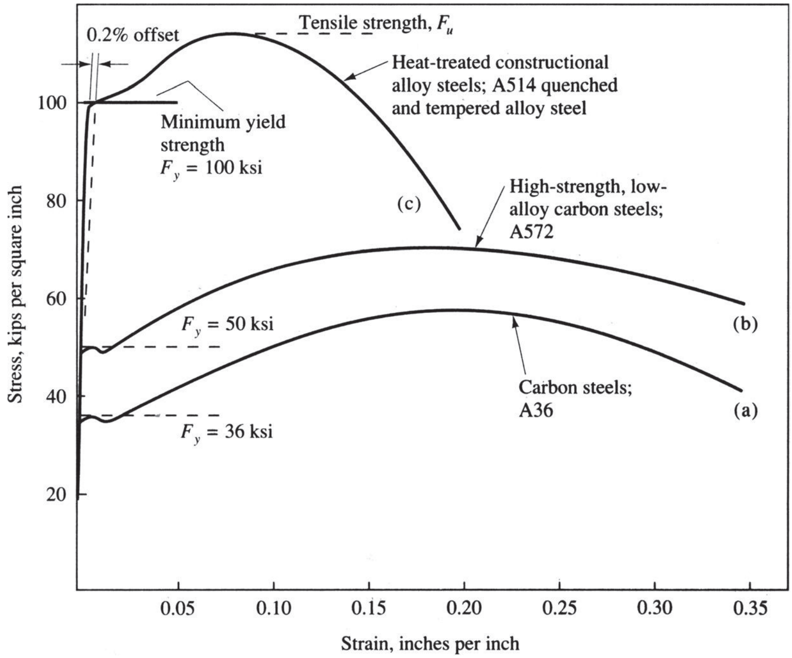

FIGURE 1.5

Typical stress–strain curves. (Based on a figure from Salmon C. G. and J. E. Johnson, *Steel Structures: Design and Behavior, Fourth Edition*. Upper Saddle River, NJ: Prentice Hall, 1996.)

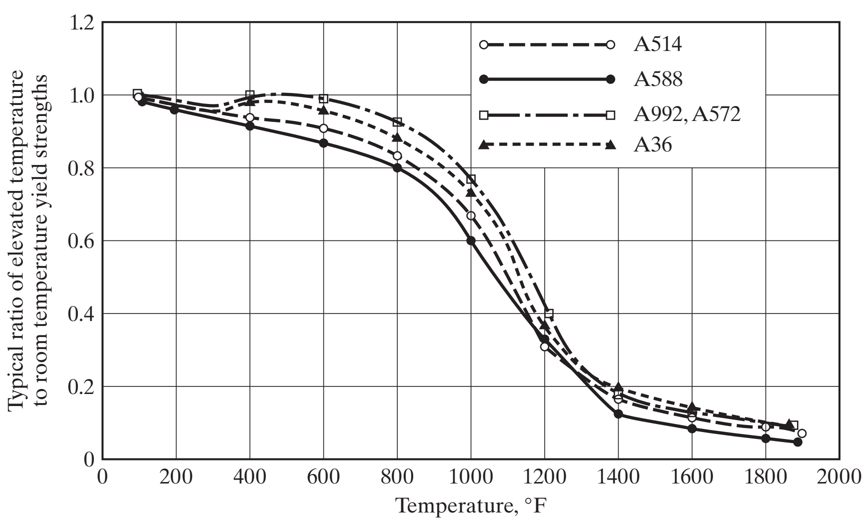

FIGURE 1.6

Effect of temperature on yield strengths.

other elements. Carbon is the material that has the greatest effect on the properties of steel. The hardness and strength of steel increase as the carbon content is increased. A 0.01 percent increase in carbon content will cause steel's yield strength to go up about 0.5 kips per square inch (ksi). Unfortunately, however, more carbon will cause steel to become more brittle and will adversely affect its weldability. If the carbon content is reduced, the steel will be softer and more ductile, but also weaker. The addition of elements such as chromium, silicon, and nickel produces steels with considerably higher strengths. Though frequently quite useful, these steels are appreciably more expensive and often are not as easy to fabricate.

A typical stress–strain diagram for a brittle steel is shown in Fig. 1.7. Unfortunately, low ductility, or brittleness, is a property usually associated with high strengths in steels (although not entirely confined to high-strength steels). As it is desirable to have both high strength and ductility, the designer may need to decide between the two extremes or to compromise. A brittle steel may fail suddenly and without warning when over-stressed and during erection could possibly fail due to the shock of erection procedures.

Brittle steels have a considerable range in which stress is proportional to strain, but do not have clearly defined yield stresses. Yet, to apply many of the formulas given in structural steel design specifications, it is necessary to have definite yield stress values, regardless of whether the steels are ductile or brittle.

If a steel member is strained beyond its elastic limit and then unloaded, it will not return to a condition of zero strain. As it is unloaded, its stress–strain diagram will follow a new path (shown in Fig. 1.7 by the dotted line parallel to the initial straight line). The result is a permanent, or residual, strain.

The line representing the stress–strain ratio for quenched and tempered steels gradually varies from a straight line so that a distinct yield point is not available. For such steels the yield stress is usually defined as the stress at the point of unloading, which corresponds to some arbitrarily defined residual strain (0.002 being the common value). In other words, we increase the strain by a designated amount and from that point draw a line parallel to the straight-line portion of the stress–strain diagram, until

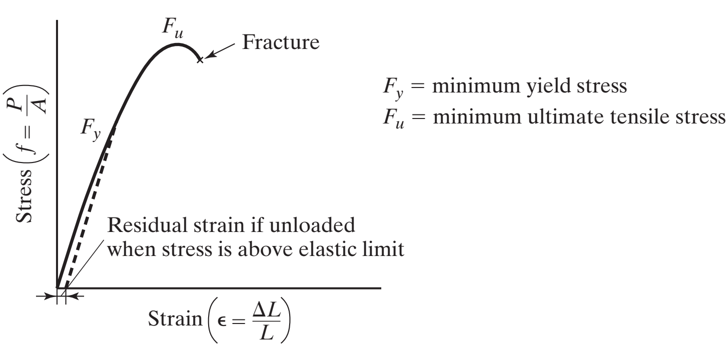

FIGURE 1.7

Typical stress–strain diagram for a brittle steel.

the new line intersects the old. This intersection is the yield stress at that particular strain. If 0.002 is used, the intersection is usually referred to as the yield stress at 0.2 percent offset strain.

1.8 MODERN STRUCTURAL STEELS

The properties of steel can be greatly changed by varying the quantities of carbon present and by adding other elements such as silicon, nickel, manganese, and copper. A steel that has a significant amount of the latter elements is referred to as an *alloy steel*. Although these elements do have a great effect on the properties of steel, the actual quantities of carbon or other alloying elements are quite small. For instance, the carbon content of steel is almost always less than 0.5 percent by weight and is normally from 0.2 to 0.3 percent.

One-half of a 170-ft clear span roof truss for the Athletic and Convention Center, Lehigh University, Bethlehem, PA. (Courtesy of National Museum of Industrial History.)

The chemistry of steel is extremely important in its effect on properties of the steel such as weldability, corrosion resistance, resistance to brittle fracture, and so on. The ASTM specifies the exact maximum percentages of carbon, manganese, silicon, and so on that are permissible for a number of structural steels. Although the physical and mechanical properties of steel sections are primarily determined by their chemical composition, they are also influenced to a certain degree by the rolling process and by their stress history and heat treatment.

In the past few decades, a structural carbon steel designated as A36 and having a minimum yield stress $F_y = 36$ ksi was the commonly used structural steel. More recently, however, most of the structural steel used in the United States is manufactured by melting scrap steel in electric furnaces. With this process, a 50 ksi steel, A992, can be produced and sold at almost the same price as 36 ksi steel.

The 50 ksi steels are the predominant ones in use today. In fact, some of the steel mills charge extra for W sections if they are to consist of A36 steel. On the other hand, 50 ksi angles have on occasion been rather difficult to obtain without special orders to the steel mills. As a result, A36 angles are still frequently used. In addition, 50 ksi plates may cost more than A36 steel.

In recent decades, the engineering and architecture professions have been continually requesting increasingly stronger steels—steels with more corrosion resistance, steels with better welding properties, and various other requirements. Research by the steel industry during this period has supplied several groups of new steels that satisfy many of the demands. Today there are quite a few structural steels designated by the ASTM and included in the AISC Specification.

Circular dome steel framed roof system.
(Ruslan Gilmanshin/123RF.)

Structural steels are generally grouped into several major ASTM classifications: the carbon steels A36, A53, A500, A501, A529, and A1085; the high-strength low-alloy steels A572, A618, A913, and A992; and the corrosion-resistant high-strength low-alloy steels A588, A847, and A1065. Considerable information is presented for each of these steels in Part 2 of the Manual. The sections that follow include a few general remarks about these steel classifications.

1.8.1 Carbon Steels

These steels as their principal strengthening agents have carefully controlled quantities of carbon and manganese. Carbon steels have their contents limited to the following maximum percentages: 1.7 percent carbon, 1.65 percent manganese, 0.60 percent silicon, and 0.60 percent copper. These steels are divided into four categories, depending on carbon percentages:

1. Low-carbon steel: < 0.15 percent.
2. Mild steel: 0.15 to 0.29 percent. (The structural carbon steels fall into this category.)
3. Medium-carbon steel: 0.30 to 0.59 percent.
4. High-carbon steel: 0.60 to 1.70 percent.

1.8.2 High-Strength Low-Alloy Steels

There are a large number of high-strength low-alloy steels, and they are included under several ASTM numbers. In addition to containing carbon and manganese, these steels owe their higher strengths and other properties to the addition of one or more alloying agents such as columbium, vanadium, chromium, silicon, copper, and nickel. Included are steels with yield stresses as low as 42 ksi and as high as 70 ksi. These steels generally have much greater atmospheric corrosion resistance than that of carbon steels.

The term *low-alloy* is used arbitrarily to describe steels for which the total of all the alloying elements does not exceed 5 percent of the total composition of the steel.

1.8.3 Atmospheric Corrosion-Resistant High-Strength Low-Alloy Structural Steels

When steels are alloyed with small percentages of copper, they become more corrosion-resistant. When exposed to the atmosphere, the surfaces of these steels oxidize and form a very tightly adherent film (sometimes referred to as a "tightly bound patina" or "a crust of rust"), which prevents further oxidation and thus eliminates the need for painting. After this process takes place (within 18 months to 3 years, depending on the type of exposure—rural, industrial, direct or indirect sunlight, etc.), the steel reaches a deep reddish-brown or black color.

Supposedly, the first steel of this type was developed in 1933 by the U.S. Steel Corporation to provide resistance to the severe corrosive conditions of railroad coal cars.

You can see the many uses that can be made of such a steel, particularly for structures with exposed members which are difficult to paint—bridges, electrical transmission towers, and others. This steel is not considered to be satisfactory for use where it is frequently subject to saltwater sprays or fogs, or continually submerged in water (fresh or salt) or the ground, or where there are severe corrosive industrial fumes. It is also not satisfactory in very dry areas, as in some western parts of the United States. For the patina to form, the steel must be subjected to a wetting and drying cycle. Otherwise, it will continue to look like unpainted steel.

Table 1.1 herein, which is Table 2-4 in the Steel Manual, lists the 12 ASTM steels mentioned earlier in this section, together with their specified minimum yield strengths (F_y) and their specified minimum tensile strengths (F_u). In addition, the right-hand columns of the table provide information regarding the availability of the shapes in the various grades of steels, as well as the preferred grade to use for each of them. The preferred steel in each case is shown with a black box.

You will note by the blackened boxes in the table that A36 is the preferred steel to be used for M, S, HP, C, MC, and L sections, while A992 is the preferred material for the most common shapes, the Ws. The lightly shaded boxes in the table refer to the shapes available in grades of steel other than the preferred grades. Before shapes are specified in these grades, the designer should check on their availability from the steel producers. Finally, the blank, or white, boxes indicate the grades of steel that are not available for certain shapes. Similar information is provided for plates and bars in Table 2-5 of the Steel Manual.

As stated previously, steels may be made stronger by the addition of special alloys. Another factor affecting steel strengths is thickness. The thinner steel is rolled, the stronger it becomes. Thicker members tend to be more brittle, and their slower cooling rates cause the steel to have a coarser microstructure.

Referring back to Table 1.1, you can see that several of the steels listed are available with different yield and tensile stresses with the same ASTM number. For instance, A572 shapes are available with 42, 50, 55, 60, and 65 ksi yield strengths. Next, reading the footnotes in Table 1.1, we note that Grades 60 and 65 steels have the footnote letter "e" by them. This footnote indicates that the only A572 shapes available with these strengths are those thinner ones which have flange thicknesses $\leq$ 2 in. Similar situations are shown in the table for several other steels, including A992 and A242.

1.9 USES OF HIGH-STRENGTH STEELS

There are indeed other groups of high-strength steels, such as the ultra-high-strength steels that have yield strengths from 160 to 300 ksi. These steels have not been included in the Steel Manual because they have not been assigned ASTM numbers.

It is said that today more than 200 steels exist in the market, which that provide yield stresses in excess of 36 ksi. The steel industry is now experimenting with steels with yield stresses varying from 200 to 300 ksi, and this may be only the beginning. Many people in the steel industry feel that steels with 500 ksi yield strengths will be made available within a few years. The theoretical binding force between iron atoms has been estimated to be in excess of 4000 ksi.[7]

[7]"L. S. Beedle et al., *Structural Steel Design* (New York: Ronald Press, 1964), p. 44.

TABLE 1.1 Applicable ASTM Specifications for Various Structural Shapes

Table 2-4
Applicable ASTM Specifications for Various Structural Shapes

Steel Typ	ASTM Designatio		F_y Yield Stress[a] (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series									
					W	M	S	HP	C	MC	L	HSS Rect.	HSS Round	Pipe
Carbon	A36		36	58–80[b]	▨	■	■	▨	■	■	■			
	A53 Gr. B		35	60										■
	A500	Gr. B	42	58									▨	
			46	58								▨		
		Gr. C	46	62									■	
			50	62								■		
	A501	Gr. A	36	58								▨	▨	
		Gr. B	50	70								▨	▨	
	A529[c]	Gr. 50	50	65–100	▨	▨	▨	▨	▨	▨	▨			
		Gr. 55	55	70–100	▨	▨	▨	▨	▨	▨	▨			
	A709	36	36	58–80[b]	▨	▨	▨	▨	▨	▨	▨			
	A1043[d,k]	36	36–52	58	▨									
		50	50–65	65	▨									
	A1085	Gr.A	50	65								▨	▨	
High-Strength Low-Alloy	A572	Gr.42	42	60	▨	▨	▨	▨	▨	▨	▨			
		Gr.50	50	65	▨	▨	▨	■	▨	▨	▨			
		Gr.55	55	70	▨	▨	▨	▨	▨	▨	▨			
		Gr. 60[e]	60	75	▨	▨	▨	▨	▨	▨	▨			
		Gr. 65[e]	65	80	▨	▨	▨	▨	▨	▨	▨			
	A618[f]	Gr. Ia[k], Ib & II	50[g]	70[g]								▨	▨	
		Gr. III	50	65								▨	▨	
	A709	50	50	65	▨	▨	▨	▨	▨	▨	▨			
		50S	50–65	65	▨	▨	▨	▨	▨	▨	▨			
		50W	50	70	▨	▨	▨	▨	▨	▨	▨			
	A913	50	50[h]	65[h]	▨	▨	▨	▨	▨	▨	▨			
		60	60	75	▨	▨	▨	▨	▨	▨	▨			
		65	65	80	▨	▨	▨	▨	▨	▨	▨			
		70	70	90	▨	▨	▨	▨	▨	▨	▨			
	A992		50[i]	65[i]	■	▨	▨	▨	▨	▨	▨			
	A1065[k]	Gr. 50[j]	50	60								▨		

■ = Preferred material specification.
▨ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

Footnotes on facing page.

Table 2-4 (continued)
Applicable ASTM Specifications for Various Structural Shapes

Steel Typ	ASTM Designatio		F_y Yield Stress[a] (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series									
												HSS		
					W	M	S	HP	C	MC	L	Rect.	Round	Pipe
Corrosion Resistant High-Strength Low-Alloy	A588				░	░	░	░	░	░	░			
	A847[k]		50	70								░	░	
	A1065[k]	Gr. 50W[j]	50	70								░		

■ = Preferred material specification.
░ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

[a] Minimum, unless a range is shown.
[b] For wide-flange shapes with flange thicknesses over 3 in., only the minimum of 58 ksi applies.
[c] For shapes with a flange or leg thickness less than or equal to 1 ½ in. only. To improve weldability, a maximum carbon equivalent can be specified (per ASTM A529 Supplementary Requirement S78). If desired, maximum tensile stress of 90 ksi can be specified (per ASTM A529 Supplementary Requirement S79).
[d] For shape profiles with a flange width of 6 in. or greater.
[e] For shapes with a flange thickness less than or equal to 2 in. only.
[f] ASTM A618 can also be specified as corrosion-resistant; see ASTM A618.
[g] Minimum applies for walls nominally ¾ in. thick and under. For wall thickness over ¾ in., F_y = 46 ksi and F_u = 67 ksi.
[h] If desired, maximum yield stress of 65 ksi and maximum yield-to-tensile strength ratio of 0.85 can be specified (per ASTM A913 Supplementary Requirement S75).
[i] A maximum yield-to-tensile strength ratio of 0.85 and carbon equivalent formula are included as mandatory, and some variation is allowed, including for shapes tested with coupons cut from the web; see ASTM A992. If desired, maximum tensile stress of 90 ksi can be specified (per ASTM A992 Supplementary Requirement S79).
[j] The grades of ASTM A1065 may not be interchanged without approval of the purchaser.
[k] This specification is not a prequalified base metal per AWS D1.1/D1.1M:2015.

Source: AISC Manual, Table 2-4, p. 2–48, 15th ed., 2017.

Although the prices of steels increase with increasing yield stresses, the percentage of price increase does not keep up with the percentage of yield-stress increase. The result is that the use of the stronger steels will quite frequently be economical for tension members, beams, and columns. Perhaps the greatest economy can be realized with tension members (particularly those without bolt holes). They may provide a great deal of savings for beams if deflections are not important or if deflections can be controlled (by methods described in later chapters). In addition, considerable economy can frequently be achieved with high-strength steels for short- and medium-length stocky columns. Another application that can provide considerable savings is hybrid construction. In this type of construction two or more steels of different strengths are used, the weaker steels being used where stresses are smaller and the stronger steels where stresses are higher.

Among the other factors that might lead to the use of high-strength steels are the following:

1. Superior corrosion resistance.
2. Possible savings in shipping, erection, and foundation costs caused by weight saving.
3. Use of shallower beams, permitting smaller floor depths.
4. Possible savings in fireproofing because smaller members can be used.

The first thought of most engineers in choosing a type of steel is the direct cost of the members. Such a comparison can be made quite easily, but determining which strength grade is most economical requires consideration of weights, sizes, deflections, maintenance, and fabrication. To make an accurate general comparison of the steels is probably impossible—rather, it is necessary to consider the specific job.

1.10 MEASUREMENT OF TOUGHNESS

The fracture toughness of steel is used as a general measure of its impact resistance, or its ability to absorb sudden increases in stress at a notch. The more ductile steel is, the greater will be its toughness. On the other hand, the lower its temperature, the higher will be its brittleness.

There are several procedures available for estimating notch toughness, but the Charpy V-notch test is the most commonly used. Although this test (which is described in ASTM Specification A6) is somewhat inaccurate, it does help identify brittle steels. With this test, the energy required to fracture a small bar of rectangular cross section with a specified notch (see Fig. 1.8) is measured.

The bar is fractured with a pendulum swung from a certain height. The amount of energy needed to fracture the bar is determined from the height to which the pendulum rises after the blow. The test may be repeated for different temperatures and the fracture energy plotted as a graph, as shown in Fig. 1.9. Such a graph clearly shows the relationship among temperature, ductility, and brittleness. The temperature at the point of steepest slope is referred to as the *transition temperature*.

Although the Charpy V-notch test is well known, it actually provides a very poor measurement. Other methods for measuring the toughness of steel are considered in articles by Barsom and Rolfe.[8,9]

Different structural steels have different specifications for required absorbed energy levels (say, 20 ft-lb at 20°F), depending on the temperature, stress, and loading conditions under which they are to be used. The topic of brittleness is continued in the next section.

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[8] J. M. Barsom, "Material Considerations in Structural Steel Design," *Engineering Journal*, AISC, 24, 3 (3rd Quarter 1987), pp. 127–139.

[9] S. T. Rolfe, "Fracture and Fatigue Control in Steel Structures," *Engineering Journal*, AISC, 14, 1 (1st Quarter 1977), pp. 2–15.

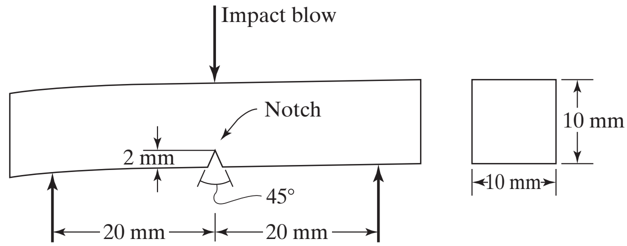

FIGURE 1.8

Specimen for Charpy V-notch test.

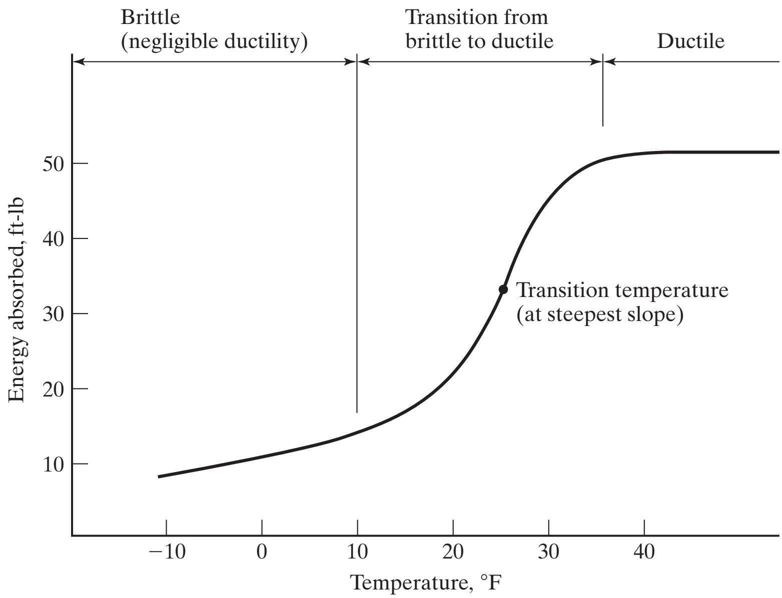

FIGURE 1.9

Results of Charpy V-notch test.

1.11 JUMBO SECTIONS

Certain heavy W sections with flange thicknesses exceeding 2 in are often referred to as *jumbo sections*. They are identified with footnotes in the W shape, Table 1.1 of the Steel Manual.

Jumbo sections were originally developed for use as compression members and are quite satisfactory for that purpose. However, designers have frequently used them for tension or flexural members. In these applications, flange and web areas have had some serious cracking problems where welding or thermal cutting has been used. These cracks may result in smaller load-carrying capacities and problems related to fatigue.[10]

Thick pieces of steel tend to be more brittle than thin ones. Some of the reasons for this are that the core areas of thicker shapes (shown in Fig. 1.10) are subject to less rolling, have higher carbon contents (needed to produce required yield stresses), and have higher tensile stresses from cooling. These topics are discussed in later chapters.

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[10]R. Bjorhovde, "Solutions for the Use of Jumbo Shapes," *Proceedings 1988 National Steel Construction Conference*, AISC, Chicago, June 8–11, pp. 2–1 to 2–20.

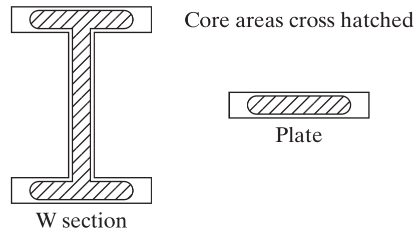


FIGURE 1.10

Core areas where brittle failure may be a problem in thick heavy members.

Jumbo sections spliced with welds can be satisfactorily used for axial tension or flexural situations if the procedures listed in Specification A3.1c of the AISC Specification are carefully followed. Included among the requirements are the following:

1. The steel used must have certain absorbed energy levels, as determined by the Charpy V-notch test (20 ft-lb at a maximum temperature of 70°F). It is absolutely necessary that the tests should be made on specimens taken from the core areas (shown in Fig. 1.10), where brittle fracture has proved to be a problem.
2. Temperature must be controlled during welding, and the work must follow a certain sequence.
3. Special splice details are required.

1.12 LAMELLAR TEARING

The steel specimens used for testing and developing stress–strain curves usually have their longitudinal axes in the direction that the steel was rolled. Should specimens be taken with their longitudinal axes transverse to the rolling direction "through the thickness" of the steel, the results will be lower ductility and toughness. Fortunately, this is a matter of little significance for almost all situations. It can, however, be quite important when thick plates and heavy structural shapes are used in highly restrained welded joints. (It can be a problem for thin members, too, but it is much more likely to give trouble in thick members.)

If a joint is highly restrained, the shrinkage of the welds in the through-the-thickness direction cannot be adequately redistributed, and the result can be a tearing of the steel called *lamellar tearing*. (*Lamellar* means "consisting of thin layers.") The situation is aggravated by the application of external tension. Lamellar tearing may show up as fatigue cracking after a number of cycles of load applications.

The lamellar tearing problem can be eliminated or greatly minimized with appropriate weld details and weld procedures. For example, the welds should be detailed so that shrinkage occurs as much as possible in the direction the steel was rolled. Several steel companies produce steels with enhanced through-the-thickness properties that provide much greater resistance to lamellar tearing. Even if such steels are used for heavy restrained joints, the special joint details mentioned before still are necessary.[11]

[11]"Commentary on Highly Restrained Welded Connections," *Engineering Journal*, AISC, vol. 10, no. 3 (3rd quarter, 1973), pp. 61–73.

Figures 8-17 and 8-18 in the Steel Manual show preferred welded joint arrangements that reduce the possibility of lamellar tearing. Further information on the subject is provided in the ASTM A770 specification.

1.13 FURNISHING OF STRUCTURAL STEEL

The furnishing of structural steel consists of the rolling of the steel shapes, the fabrication of the shapes for the particular job (including cutting to the proper dimensions and punching the holes necessary for field connections), and their erection. Very rarely will a single company perform all three of these functions, and the average company performs only one or two of them. For instance, many companies fabricate structural steel and erect it, while others may be only steel fabricators or steel erectors. There are approximately 400 to 500 companies in the United States that make up the fabricating industry for structural steel. Most of them do both fabrication and erection.

Steel fabricators normally carry very little steel in stock because of the high interest and storage charges. When they get a job, they may order the shapes to certain lengths directly from the rolling mill, or they may obtain them from service centers. Service centers, which are an increasingly important factor in the supply of structural steel, buy and stock large quantities of structural steel, which they buy at the best prices they can find anywhere in the world.

Structural steel is usually designed by an engineer in collaboration with an architectural firm. The designer makes design drawings that show member sizes, controlling dimensions, and any unusual connections. The company that is to fabricate the steel makes the detailed drawings subject to the engineer's approval. These drawings provide all the information necessary to fabricate the members correctly. They show the dimensions for each member, the locations of holes, the positions and sizes of connections, and the like. A part of a typical detail drawing for a bolted steel beam is shown in Fig. 1.11. There may be a few items included on this drawing that are puzzling to you since you have read only a few pages of this book. However, these items should become clear as you study the chapters to follow.

On actual detail drawings, details probably will be shown for several members. Here, the author has shown only one member, just to indicate the information needed so that the shop can correctly fabricate the member. The darkened circles and rectangles indicate that the bolts are to be installed in the field, while the nondarkened ones show the connections which are to be made in the shop.

The erection of steel buildings is more a matter of assembly than nearly any other part of construction work. Each of the members is marked in the shop with letters and numbers to distinguish it from the other members to be used. The erection is performed in accordance with a set of erection plans. These plans are not detailed drawings, but are simple line diagrams showing the position of the various members in the building. The drawings show each separate piece or subassembly of pieces together with assigned shipping or erection marks, so that the steelworkers can quickly identify and locate members in their correct positions in the structure. (Persons performing steel erection often are called *ironworkers*, which is a name held over from the days before structural steel.) Directions (north, south, east, or west) usually are painted on column faces.

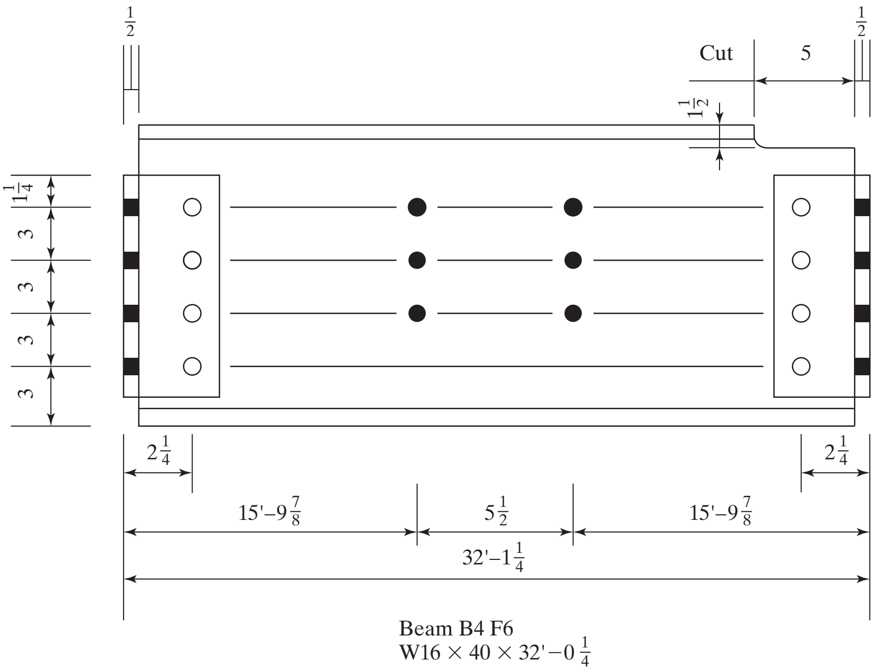
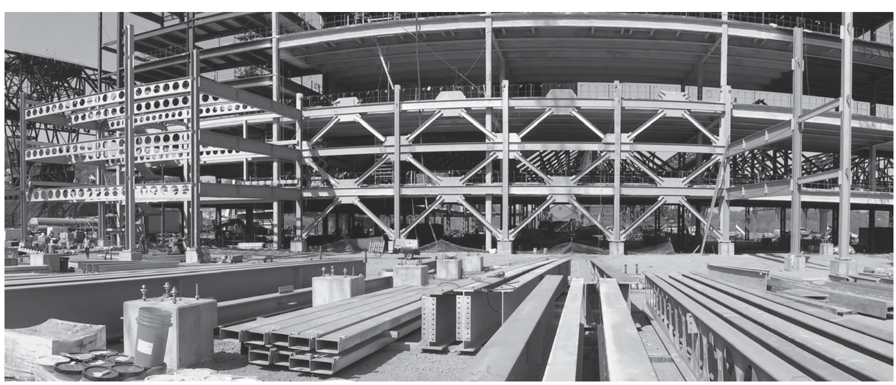


FIGURE 1.11

Part of a detail drawing.

Sometimes, the erection drawing gives the sizes of the members, but this is not necessary. They may or may not be shown, depending on the practice of the particular fabricator.

Beams, girders, and columns will be indicated on the drawings by the letters B, G, or C, respectively, followed by the number of the particular member as B5, G12, and so on. Often, there will be several members of these same designations where members are repeated in the building.



Beth Sands Project, Bethlehem, PA. (Courtesy of CMC South Carolina Steel.)

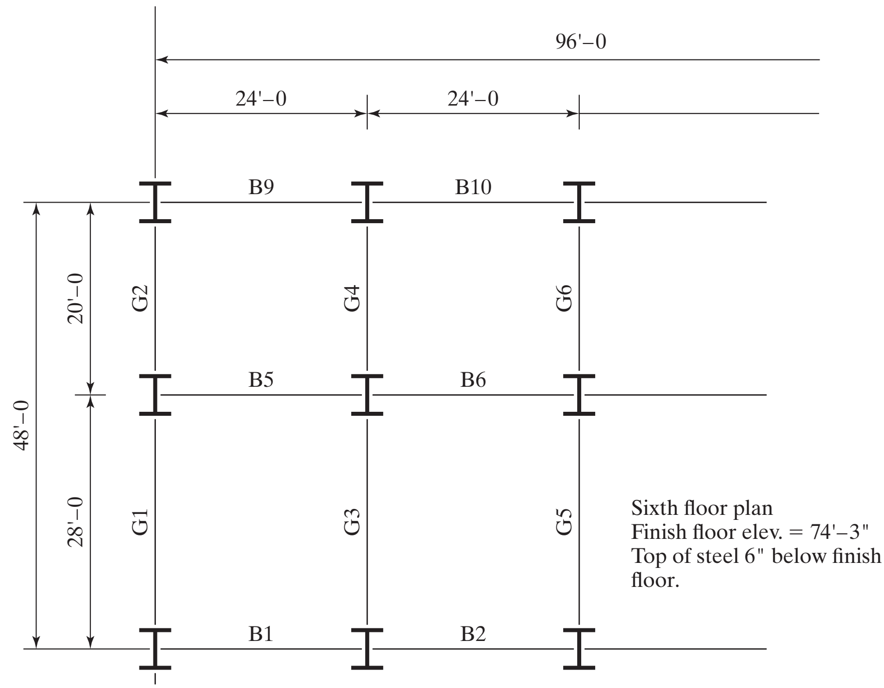


FIGURE 1.12

Part of an erection drawing showing where each member is to be located.

Multistory steel frames often will have several levels of identical or nearly identical framing systems. Thus, one erection plan may be used to serve several floors. For such situations, the member designations for the columns, beams, and girders will have the level numbers incorporated in them. For instance, column C15(3–5) is column 15, third to fifth floors, while B5F6, or just B5(6), represents beam B5 for the sixth floor. A portion of a building erection drawing is shown in Fig. 1.12.

Next, we describe briefly the erection of the structural steel members for a building. Initially, a group of ironworkers, sometimes called the "raising gang," erects the steel members, installing only a sufficient number of bolts to hold the members in position. In addition, they place any guy cables where needed for stability and plumbing of the steel frame.

Another group of ironworkers, who are sometimes referred to as the "detail gang," install the remaining bolts, carry out any needed field welding, and complete the plumbing of the structure. After the last two steps are completed, another crew installs the metal decking for the floor and roof slabs. They in turn are followed by the crews who place the necessary concrete reinforcing and the concrete for those slabs.[12]

[12]A. R. Tamboli, editor, *Steel Design Handbook LRFD Method* (New York: McGraw-Hill, 1997), pp. 12–37.

1.14 THE WORK OF THE STRUCTURAL DESIGNER

The structural designer arranges and proportions structures and their parts so that they will satisfactorily support the loads to which they may feasibly be subjected. It might be said that he or she is involved with the general layout of structures; studies of the possible structural forms that can be used; consideration of loading conditions; analysis of stresses, deflections, and so on; design of parts; and the preparation of design drawings. More precisely, the word *design* pertains to the proportioning of the various parts of a structure after the forces have been calculated, and it is this process which will be emphasized throughout the text, using structural steel as the material.

1.15 RESPONSIBILITIES OF THE STRUCTURAL DESIGNER

The structural designer must learn to arrange and proportion the parts of structures so that they can be practically erected and will have sufficient strength, serviceability, and reasonable economy. These items are discussed briefly next.

1.15.1 Safety

Not only must the frame of a structure safely support the loads to which it is subjected (strength), but it must support them in such a manner that deflections and vibrations are not so great as to frighten the occupants or to cause unsightly cracks (serviceability).

1.15.2 Cost

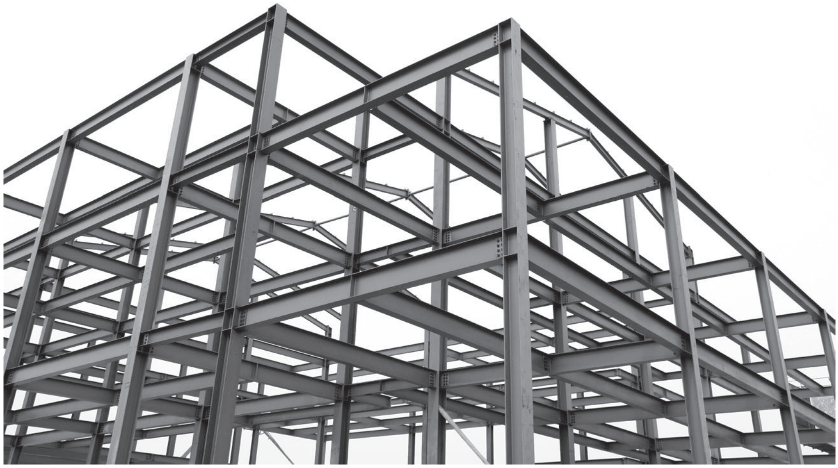
The designer needs to keep in mind the factors that can lower cost without sacrifice of strength. These items, which are discussed in more detail throughout the text, include the use of standard-size members, simple connections and details, and members and materials that will not require an unreasonable amount of maintenance through the years.

1.15.3 Constructability

Another objective is the design of structures that can be fabricated and erected without great problems arising. Designers need to understand fabrication methods and should try to fit their work to the fabrication facilities available.

Designers should learn everything possible about the detailing, fabrication, and field erection of steel. The more the designer knows about the problems, tolerances, and clearances in shop and field, the more probable it is that reasonable, practical, and economical designs will be produced. This knowledge should include information concerning the transportation of the materials to the job site (such as the largest pieces that can be transported practically by rail or truck), labor conditions, and the equipment available for erection. Perhaps the designer should ask, "Could I get this thing together if I were sent out to do it?"

Finally, he or she needs to proportion the parts of the structure so that they will not unduly interfere with the mechanical features of the structure (pipes, ducts, etc.) or the architectural effects.



Steel framed office building during construction. (Wang Aizhong/123RF.)

1.16 ECONOMICAL DESIGN OF STEEL MEMBERS

The design of a steel member involves much more than a calculation of the properties required to support the loads and the selection of the lightest section providing these properties. Although at first glance this procedure would seem to give the most economical designs, many other factors need to be considered.

Today, the labor costs involved in the fabrication and erection of structural steel are thought to run close to 60 percent of the total costs of steel structures. On the other hand, material costs represent only about 25 percent of total costs. Thus, we can see that any efforts we make to improve the economy of our work in structural steel should be primarily concentrated in the labor area.

When designers are considering costs, they have a tendency to think only of quantities of materials. As a result, they will sometimes carefully design a structure with the lightest possible members and end up with some very expensive labor situations with only minor material savings. Among the many factors that need to be considered in providing economical steel structures are the following:

1. One of the best ways to achieve economy is to have open communications between designers, fabricators, erectors, and others involved in a particular project. If this is done during the design process, the abilities and experience of each of the parties may be utilized at a time when it is still possible to implement good economical ideas.
2. The designer needs to select steel sections of sizes that are usually rolled. Steel beams and bars and plates of unusual sizes will be difficult to obtain during boom periods and will be expensive during any period. A little study on the designer's

part will enable him or her to avoid these expensive shapes. Steel fabricators are constantly supplied with information from the steel companies and the steel warehousers as to the sizes and lengths of sections available. (Most structural shapes can be procured in lengths from 60 to 75 ft, depending on the producer, while it is possible under certain conditions to obtain some shapes up to 120 ft in length.)

3. A blind assumption that the lightest section is the cheapest one may be in considerable error. A building frame designed by the "lightest-section" procedure will consist of a large number of different shapes and sizes of members. Trying to connect these many-sized members and fit them in the building will be quite complicated, and the pound price of the steel will, in all probability, be rather high. A more reasonable approach would be to smooth out the sizes by selecting many members of the same sizes, although some of them may be slightly overdesigned.
4. The beams usually selected for the floors in buildings will normally be the deeper sections, because these sections for the same weights have the largest moments of inertia and the greatest resisting moments. As building heights increase, however, it may be economical to modify this practice. As an illustration, consider the erection of a 20-story building, for which each floor has a minimum clearance. It is assumed that the depths of the floor beams may be reduced by 6 in without an unreasonable increase in beam weights. The beams will cost more, but the building height will be reduced by $20 \times 6 \text{ in} = 120 \text{ in}$, or 10 ft, with resulting savings in walls, elevator shafts, column heights, plumbing, wiring, and footings.[13]
5. The costs of erection and fabrication for structural steel beams are approximately the same for light and heavy members. Thus, beams should be spaced as far apart as possible to reduce the number of members that have to be fabricated and erected.
6. Structural steel members should be painted only if so required by the applicable specification. You should realize that steel should not be painted if it is to be in contact with concrete. Furthermore, the various fireproofing materials used for protecting steel members adhere better if the surfaces are unpainted.[14]
7. It is very desirable to keep repeating the same section over and over again. Such a practice will reduce the detailing, fabrication, and erection costs.
8. For larger sections, particularly the built-up ones, the designer needs to have information pertaining to transportation problems. The desired information includes the greatest lengths and depths that can be shipped by truck or rail (see Section 1.18), clearances available under bridges and power lines leading to the project, and allowable loads on bridges. It may be possible to fabricate a steel

[13]H. Allison, "Low- and Medium-Rise Steel Buildings," (Chicago: AISC, 1991), pp. 1–5.

[14]Ibid, pp. 1–5.

roof truss in one piece, but is it possible to transport it to the job site and erect it in one piece?

9. Sections should be selected that are reasonably easy to erect and which have no conditions that will make them difficult to maintain. As an example, it is necessary to have access to all exposed surfaces of steel bridge members so that they may be periodically painted (unless one of the special corrosion-resistant steels is used).
10. Buildings are often filled with an amazing conglomeration of pipes, ducts, conduits, and other items. Every effort should be made to select steel members that will fit in with the requirements made by these items.
11. The members of a steel structure are often exposed to the public, particularly in the case of steel bridges and auditoriums. Appearance may often be the major factor in selecting the type of structure, such as where a bridge is desired that will fit in and actually contribute to the appearance of an area. Exposed members may be surprisingly graceful when a simple arrangement, perhaps with curved members, is used, but other arrangements may create a terrible eyesore. The student has certainly seen illustrations of each case. It is very interesting to know that beautiful structures in steel are usually quite reasonable in cost.

The question is often asked, *How do we achieve economy in structural steel design?* The answer is simple: *It lies in what the steel fabricator does not have to do*. (In other words, economy can be realized when fabrication is minimized.)

The April 2000 issue of *Modern Steel Construction* provides several articles which present excellent material on the topic of economy in steel construction.[15] The student can very quickly learn a great deal of valuable information concerning the topic of economy in steel by reading these articles. The author thinks they are a "must read" for anyone practicing steel design.[16–19]

1.17 FAILURE OF STRUCTURES

Many people who are superstitious do not discuss flat tires or make their wills, because they fear they will be tempting fate. These same people would probably not care to discuss the subject of engineering failures. Despite the prevalence of this superstition,

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[15] *Modern Steel Construction,* April 2000, vol. 40, no. 4 (Chicago: American Institute of Steel Construction), pp. 6, 25–48, 60.

[16] C. J. Carter, T. M. Murray and W. A. Thornton, "Economy in Steel," in *Modern Steel Construction*, April 2000, vol. 40, no. 4 (Chicago: American Institute of Steel Construction).

[17] D. T. Ricker, "Value Engineering for Steel Construction," in *Modern Steel Construction*, April 2000, vol. 40, no. 4 (Chicago: American Institute of Steel Construction).

[18] J. E. Quinn, "Reducing Fabrication Costs," in *Modern Steel Construction*, April 2000, vol. 40, no. 4 (Chicago: American Institute of Steel Construction).

[19] Steel Joist Institute, "Reducing Joist Cost," in *Modern Steel Construction*, April 2000, vol. 40, no. 4 (Chicago: American Institute of Steel Construction).

an awareness of the items that have most frequently caused failures in the past is invaluable to experienced and inexperienced designers alike. Perhaps a study of past failures is more important than a study of past successes. Benjamin Franklin supposedly made the observation that "a wise man learns more from failures than from success."

The designer with little experience particularly needs to know where the most attention should be given and where outside advice is needed. The vast majority of designers, experienced and inexperienced, select members of sufficient size and strength. The collapse of structures is usually due to insufficient attention to the details of connections, deflections, erection problems, and foundation settlement. Rarely, if ever, do steel structures fail due to faults in the material, but rather due to its improper use.

A frequent fault of designers is that after carefully designing the members of a structure, they carelessly select connections which may or may not be of sufficient size. They may even turn the job of selecting the connections over to drafters, who may not have sufficient understanding of the difficulties that can arise in connection design. Perhaps the most common mistake made in connection design is to neglect some of the forces acting on the connections, such as twisting moments. In a truss for which the members have been designed for axial forces only, the connections may be eccentrically loaded, resulting in moments that cause increasing stresses. These secondary stresses are occasionally so large that they need to be considered in design.

Another source of failure occurs when beams supported on walls have insufficient bearing or anchorage. Imagine a beam of this type supporting a flat roof on a rainy night when the roof drains are not functioning properly. As the water begins to form puddles on the roof, the beam tends to sag in the middle, causing a pocket to catch more rain, which creates more beam sag, and so on. As the beam deflects, it pushes out against the walls, possibly causing collapse of walls or slippage of beam ends off the wall. Picture a 60-ft steel beam, supported on a wall with only an inch or two of bearing, that contracts when the temperature drops 50 or 60 degrees overnight. A collapse due to a combination of beam contraction, outward deflection of walls, and vertical deflection caused by precipitation loads is not difficult to visualize; furthermore, actual cases in engineering literature are not difficult to find.

Foundation settlements cause a large number of structural failures, probably more than any other factor. Most foundation settlements do not result in collapse, but they very often cause unsightly cracks and depreciation of the structure. If all parts of the foundation of a structure settle equally, the stresses in the structure theoretically will not change. The designer, usually not able to prevent settlement, has the goal of designing foundations in such a manner that equal settlements occur. Equal settlements may be an impossible goal, and consideration should be given to the stresses that would be produced if settlement variations occurred. The student's background in structural analysis will tell him or her that uneven settlements in statically indeterminate structures may cause extreme stress variations. Where foundation conditions are poor, it is desirable, if feasible, to use statically determinate structures whose stresses are not appreciably changed by support settlements. (The student will learn in subsequent discussions that the ultimate strength of steel structures is usually affected only slightly by uneven support settlements.)

Long span steel framing for industrial building during construction. (Vladimir Prizemlin/123RF.)

Some structural failures occur because inadequate attention is given to deflections, fatigue of members, bracing against swaying, vibrations, and the possibility of buckling of compression members or the compression flanges of beams. The usual structure, when completed, is sufficiently braced with floors, walls, connections, and special bracing, but there are times during construction when many of these items are not present. As previously indicated, the worst conditions may well occur during erection, and special temporary bracing may be required.

1.18 HANDLING AND SHIPPING STRUCTURAL STEEL

The following general rules apply to the sizes and weights of structural steel pieces that can be fabricated in the shop, shipped to the job, and erected:

1. The maximum weights and lengths that can be handled in the shop and at a construction site are roughly 90 tons and 120 ft, respectively.
2. Pieces as large as 8 ft high, 8 ft wide, and 60 ft long can be shipped on trucks with no difficulty (provided the axle or gross weights do not exceed the permissible values given by public agencies along the designated routes).
3. There are few problems in railroad shipment if pieces are no larger than 10 ft high, 8 ft wide, and 60 ft long, and weigh no more than 20 tons.
4. Routes should be carefully studied, and carriers consulted for weights and sizes exceeding the values given in (2) and (3).

1.19 CALCULATION ACCURACY

A most important point many students with their superb pocket calculators and personal computers have difficulty understanding is that structural design is not an exact science for which answers can confidently be calculated to eight significant figures. Among the reasons for this fact are that the methods of analysis are based on partly true assumptions, the strengths of materials used vary appreciably, and maximum loadings can be only approximated. With respect to this last reason, how many of the users of this book could estimate within 10 percent the maximum load in pounds per square foot that will ever occur on the building floor which they are now occupying? Calculations to more than two or three significant figures are obviously of little value and may actually be harmful in that they mislead the student by giving him or her a fictitious sense of precision. From a practical standpoint, it seems wise to carry all the digits on the calculator for intermediate steps and then round off the final answers.

1.20 COMPUTERS AND STRUCTURAL STEEL DESIGN

The availability of personal computers has drastically changed the way steel structures are analyzed and designed. In nearly every engineering school and design office, computers are used to perform structural analysis problems. Many of the structural analysis programs commercially available also can perform structural design.

Many calculations are involved in structural steel design, and many of these calculations are quite time-consuming. With the use of a computer, the design engineer can greatly reduce the time required to perform these calculations, and likely increase the accuracy of the calculations. In turn, this will then provide the engineer with more time to consider the implications of the design and the resulting performance of the structure, and more time to try changes that may improve economy or behavior.

Although computers do increase design productivity, they also tend to reduce the engineer's "feel" for the structure. This can be a particular problem for young engineers with very little design experience. Unless design engineers have this feel for system behavior, the use of computers can result in large, costly mistakes. Such situations may arise where anomalies and inconsistencies are not immediately apparent to the inexperienced engineer. Theoretically, the computer design of alternative systems for a few projects should substantially improve the engineer's judgment in a short span of time. Without computers, the development of this same judgment would likely require the engineer to work his or her way through numerous projects.

1.21 PROBLEMS FOR SOLUTION

1-1. When did an economical production method of steel become available?

1-2. List the three regions of a stress–strain diagram for mild- or low-carbon structural steel.

1-3. What production method for steel shapes is specified by the following organizations?

- a. AISI (American Iron and Steel Institute)
- b. AISC (American Institute of Steel Construction)

1-4. Define the following:

- a. Yield stress
- b. Proportional limit
- c. Elastic limit

1-5. List the preferred steel type (ASTM specification) for the following:

- a. Angles
- b. W shape
- c. Plates

1-6. What is the range of carbon content in the following materials?

- a. Cast iron
- b. Wrought iron
- c. Steel

1-7. List the functions for furnishing of structural steel that are rarely performed by a single company.

1-8. List four advantages of steel as a structural material.

1-9. List four disadvantages of steel as a structural material.

1-10. What is the minimum yield stress of the predominant structural steel used today?

1-11. List four types of failures for structural steel structures.

1-12. What are the limiting dimensions (height, width, and length) for individual pieces of structural steel that can be shipped by trucks on highways with no difficulty or special permitting.

CHAPTER 2

Specifications, Loads, and Methods of Design

2.1 SPECIFICATIONS AND BUILDING CODES

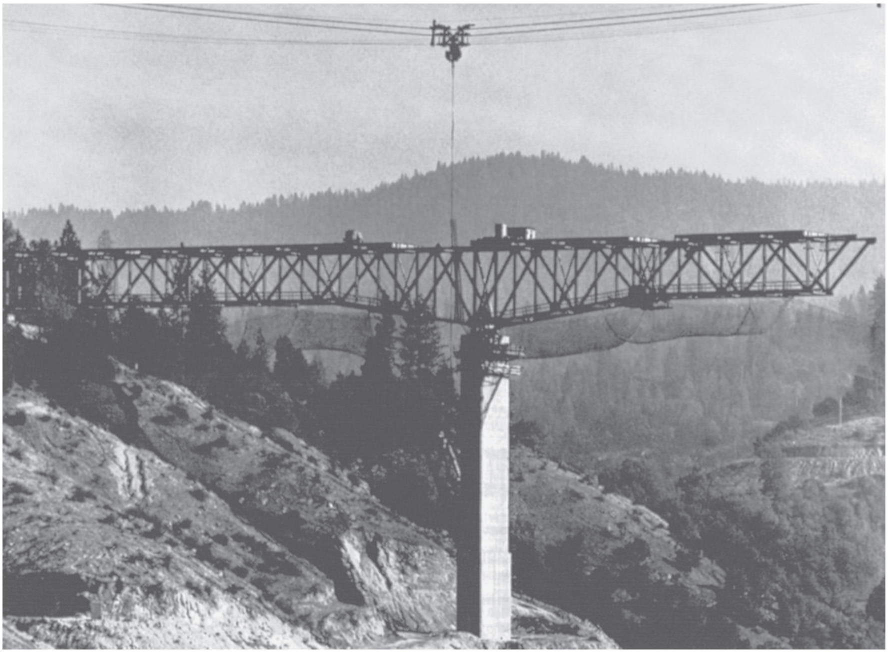
The design of most structures is controlled by building codes and design specifications. Even if they are not so controlled, the designer will probably refer to them as a guide. No matter how many structures a person has designed, it is impossible for him or her to have encountered every situation. By referring to specifications, he or she is making use of the best available material on the subject. Engineering specifications that are developed by various organizations present the best opinion of those organizations as to what represents good practice.

Municipal and state governments concerned with the safety of the public have established building codes with which they control the construction of various structures within their jurisdiction. These codes, which are actually laws or ordinances, specify minimum design loads, design stresses, construction types, material quality, and other factors. They vary considerably from city to city, a fact that causes some confusion among architects and engineers.

Several organizations publish recommended practices for regional or national use. Their specifications are not legally enforceable, unless they are embodied in the local building code or made a part of a particular contract. Among these organizations are the AISC and AASHTO (American Association of State Highway and Transportation Officials). Nearly all municipal and state building codes have adopted the AISC Specification, and nearly all state highway and transportation departments have adopted the AASHTO Specifications.

Readers should note that logical and clearly written codes are quite helpful to design engineers. Furthermore, there are far fewer structural failures in areas that have good building codes that are strictly enforced.

Some people feel that specifications prevent engineers from thinking for themselves—and there may be some basis for the criticism. They say that the ancient



South Fork Feather River Bridge in northern California, being erected by use of a 1626-ft-long cableway strung from 210-ft-high masts anchored on each side of the canyon. (Courtesy of National Museum of Industrial History.)

engineers who built the great pyramids, the Parthenon, and the great Roman bridges were controlled by few specifications, which is certainly true. On the other hand, it should be said that only a few score of these great projects have endured over many centuries, and they were, apparently, built without regard to cost of material, labor, or human life. They were probably built by intuition and by certain rules of thumb that the builders developed by observing the minimum size or strength of members, which would fail only under given conditions. Their likely numerous failures are not recorded in history; only their successes endured.

Today, however, there are hundreds of projects being constructed at any one time in the United States that rival in importance and magnitude the famous structures of the past. It appears that if all engineers in our country were allowed to design projects such as these, without restrictions, there would be many disastrous failures. *The important thing to remember about specifications, therefore, is that they are written not for the purpose of restricting engineers, but for the purpose of protecting the public.*

No matter how many specifications are written, it is impossible for them to cover every possible design situation. As a result, no matter which building code or specification is or is not being used, the ultimate responsibility for the design of a safe structure lies with the structural designer. Obviously, the intention of these specifications is that the loading used for design should be the one that causes the largest stresses.

Another very important code, the *International Building Code*[1] (IBC), was developed because of the need for a modern building code that emphasizes performance. It is intended to provide a model set of regulations to safeguard the public in all communities.

2.2 LOADS

Perhaps the most important and most difficult task faced by the structural engineer is the accurate estimation of the loads that may be applied to a structure during its life. No loads that may reasonably be expected to occur may be overlooked. After loads are estimated, the next problem is to determine the worst possible combinations of these loads that might occur at one time. For instance, would a highway bridge completely covered with ice and snow be simultaneously subjected to fast-moving lines of heavily loaded trailer trucks in every lane and to a 90-mile lateral wind, or is some lesser combination of these loads more likely?

Section B2 of the AISC Specification states that the nominal loads to be used for structural design shall be the ones stipulated by the applicable code under which the structure is being designed or as dictated by the conditions involved. If there is an absence of a code, the design loads shall be those provided in a publication of the American Society of Civil Engineers entitled *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*.[2] This publication is commonly referred to as ASCE 7. It was originally published by the American National Standards Institute (ANSI) and referred to as the *ANSI 58.1 Standard*. The ASCE took over its publication in 1988.

In general, loads are classified according to their character and duration of application. As such, they are said to be *dead loads, live loads,* and *environmental loads*. Each of these types of loads are discussed in the next few sections.

2.3 DEAD LOADS

Dead loads are loads of constant magnitude that remain in one position. They consist of the structural frame's own weight and other loads that are permanently attached to the frame. For a steel-frame building, the frame, walls, floors, roof, plumbing, and fixtures are dead loads.

To design a structure, it is necessary for the weights, or dead loads, of the various parts to be estimated for use in the analysis. The exact sizes and weights of the parts are not known until the structural analysis is made and the members of the structure selected. The weights, as determined from the actual design, must be compared with the estimated weights. If large discrepancies are present, it will be necessary to repeat the analysis and design with better estimated weights.

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[1]International Code Council, Inc., *International Building Code* (Washington, DC, 2014).

[2]American Society of Civil Engineers, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. ASCE 7-16. Formerly ANSI A58.1 (Reston, VA.:ASCE, 2016).

TABLE 2.1 Typical Dead Loads for Some Common Building Materials

Reinforced concrete	150 lb/cu ft
Structural steel	490 lb/cu ft
Plain concrete	145 lb/cu ft
Movable steel partitions	4 psf
Plaster on concrete	5 psf
Suspended steel channel system	2 psf
5-Ply felt and gravel	6 psf
Hardwood flooring (7/8 in)	4 psf
2×12 @ 16 in spacing double wood floors	7 psf
Wood studs with 1/2 in gypsum each side	8 psf
Clay brick wythes (4 in)	39 psf

Reasonable estimates of structure weights may be obtained by referring to similar types of structures or to various formulas and tables available in several publications. The weights of many materials are given in Part 17 of the Steel Manual. Even more detailed information on dead loads is provided in Tables C3-1 and C3-2 of ASCE 7-16. An experienced engineer can estimate very closely the weights of most materials and will spend little time repeating designs because of poor estimates.

The approximate weights of some common building materials for roofs, walls, floors, and so on are presented in Table 2.1.

2.4 LIVE LOADS

Live loads are loads that may change in position and magnitude. They are caused when a structure is occupied, used, and maintained. Live loads that move under their own power, such as trucks, people, and cranes, are said to be *moving loads*. Those loads that may be moved are *movable loads*, such as furniture and warehouse materials. A great deal of information on the magnitudes of these various loads, along with specified minimum values, are presented in ASCE 7-16.

1. *Floor loads*. The minimum gravity live loads to be used for building floors are clearly specified by the applicable building code. Unfortunately, however, the values given in these various codes vary from city to city, and the designer must be sure that his or her designs meet the requirements of the locality in question. A few of the typical values for floor loadings are listed in Table 2.2. These values were adopted from ASCE 7-16. In the absence of a governing code, this is an excellent one to follow.

Quite a few building codes specify concentrated loads that must be considered in design. Section 4.4 of ASCE 7-16 and Section 1607.4 of IBC-2015 are two such examples. The loads specified are considered as alternatives to the uniform loads previously considered herein.

Some typical concentrated loads taken from Table 4-1 of ASCE 7-16 and Table 1607.1 of IBC-2015 are listed in Table 2.3. These loads are to be placed on

TABLE 2.2 Typical Minimum Uniform Live Loads for Design of Buildings

Type of building	LL (psf)
Apartment houses	
Apartments	40
Public rooms	100
Dining rooms and restaurants	100
Garages (passenger cars only)	40
Gymnasiums, main floors, and balconies	100
Office buildings	
Lobbies	100
Offices	50
Schools	
Classrooms	40
Corridors, first floor	100
Corridors above first floor	80
Storage warehouses	
Light	125
Heavy	250
Stores (retail)	
First floor	100
Other floors	75

TABLE 2.3 Typical Concentrated Live Loads for Buildings

Hospitals—operating rooms, private rooms, and wards	1000 lb
Manufacturing building (light)	2000 lb
Manufacturing building (heavy)	3000 lb
Office floors	2000 lb
Retail stores (first floors)	1000 lb
Retail stores (upper floors)	1000 lb
School classrooms	1000 lb
School corridors	1000 lb

floors or roofs at the positions where they will cause the most severe conditions. Unless otherwise specified, each of these concentrated loads is spread over an area 2.5×2.5 ft square (6.25 ft^2).

2. *Traffic loads for bridges*. Bridges are subjected to series of concentrated loads of varying magnitude caused by groups of truck or train wheels.
3. *Impact loads*. Impact loads are caused by the vibration of moving or movable loads. It is obvious that a crate dropped on the floor of a warehouse or a truck bouncing on uneven pavement of a bridge causes greater forces than would occur if the loads were applied gently and gradually. Cranes picking up loads and elevators starting and stopping are other examples of impact loads. Impact loads are equal to the difference between the magnitude of the loads actually caused and the magnitude of the loads had they been dead loads.

TABLE 2.4 Live Load Impact Factors

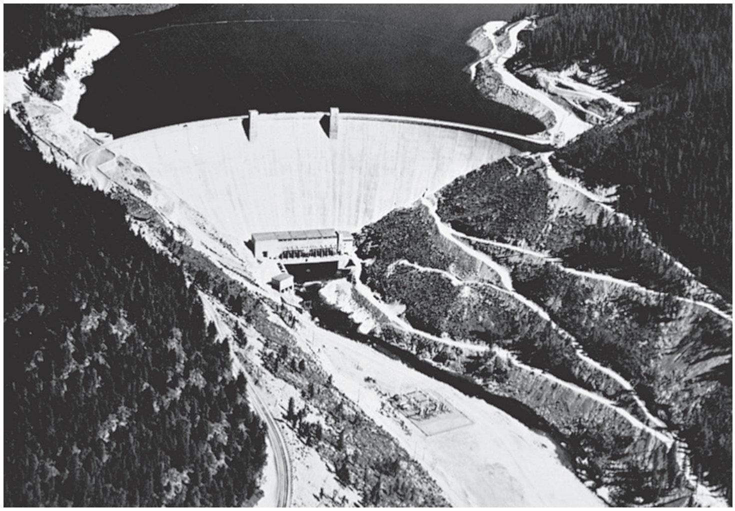
Elevator machinery*	100%
Motor-driven machinery	20%
Reciprocating machinery	50%

*See Section C4.6, ASCE 7-16 Commentary.

Section 4.6 of ASCE 7-16 Specification requires that when structures are supporting live loads that tend to cause impact, it is necessary for those loads to be increased by the percentages given in Table 2.4.

4. *Longitudinal loads.* Longitudinal loads are another type of load that needs to be considered in designing some structures. Stopping a train on a railroad bridge or a truck on a highway bridge causes longitudinal forces to be applied. It is not difficult to imagine the tremendous longitudinal force developed when the driver of a 40-ton-trailer truck traveling 60 mph suddenly has to apply the brakes while crossing a highway bridge. There are other longitudinal load situations, such as ships bumping a dock during berthing and the movement of traveling cranes that are supported by building frames.
5. *Other live loads*. Among the other types of live loads with which the structural engineer will have to contend are *soil pressures* (such as the exertion of lateral earth pressures on walls or upward pressures on foundations); *hydrostatic*

Roof/bridge crane framing, Savannah, GA. (Courtesy of CMC South Carolina Steel.)



Hungry Horse Dam and Reservoir, Rocky Mountains, in northwestern Montana. (Courtesy of the Montana Travel Promotion Division.)

pressures (water pressure on dams, inertia forces of large bodies of water during earthquakes, and uplift pressures on tanks and basement structures); *blast loads* (caused by explosions, sonic booms, and military weapons); *thermal forces* (due to changes in temperature, causing structural deformations and resulting structural forces); and *centrifugal forces* (such as those on curved bridges and caused by trucks and trains, or similar effects on roller coasters, etc.).

2.5 ENVIRONMENTAL LOADS

Environmental loads are caused by the environment in which a particular structure is located. For buildings, environmental loads are caused by rain, snow, wind, temperature change, and earthquakes. Strictly speaking, environmental loads are live loads, but they are the result of the environment in which the structure is located. Even though they do vary with time, they are not all- caused by gravity or operating conditions, as is typical with other live loads. A few comments are presented in the paragraphs that follow concerning the different types of environmental loads:

1. *Snow*. In the colder states, snow loads are often quite important. One inch of snow is equivalent to a load of approximately 0.5 psf (pounds per square foot), but it may be higher at lower elevations where snow is denser. For roof designs, snow loads varying from 10 to 40 psf are commonly used, the magnitude depending primarily on the slope of the roof and, to a lesser degree, on the character of the roof surface. The larger values are used for flat roofs, the smaller ones for sloped roofs. Snow tends to slide off sloped roofs, particularly those with metal or slate surfaces. A load of approximately 10 psf might be used for 45° slopes and a 40-psf