Chapter 1

Introduction

One World Trade Center, New York
Photo Courtesy Michael Mahesh, Port Authority of NYNJ

1.1 SCOPE

A wide variety of designs can be characterized as *structural steel design.* This book deals with the design of steel structures for buildings as governed by the *ANSI/AISC 360-16 Specification for Structural Steel Buildings*, published by the American Institute of Steel Construction (AISC) in 2016, and referred to as the *Specification* in this book. The areas of application given throughout this book specifically focus on the design of steel building structures. The treatment of subjects associated with bridges and industrial structures, if addressed at all, is kept relatively brief.

The book addresses the concepts and design criteria for the two design approaches detailed by the *Specification*: Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD). Both methods are discussed later in this chapter.

In addition to the *Specification*, the primary reference for this book is the 15th edition of the *AISC Steel Construction Manual.* This reference handbook contains tables of the basic values needed for structural steel design, design tables to simplify actual design, and the complete *Specification*. Throughout this book, this is referred to as the *Manual*.

1.2 THE SPECIFICATION

The *ANSI/AISC 360-16 Specification for Structural Steel Buildings* is the latest in a long line of standard specifications published by the American Institute of Steel Construction for the design and construction of structural steel buildings. The first edition was published in 1923. For the reader interested in the historical aspects of these specifications, AISC has two resources that provide detailed guidance on the historical structural steel standards. The first is AISC Design Guide 15, *AISC Rehabilitation and Retrofit Guide: A Reference for Historic Shapes and Specifications*. This Design Guide provides outline comparisons of the provisions in the different editions of the *Specification*. The second resource is found on the AISC web site, www.aisc.org, *AISC Specifications 1923–2010*, which contains a searchable compendium of all of the AISC *Specifications for Structural Steel Buildings* produced from 1923 through 2010.

Current design is carried out under the provisions published in the 2016 edition of the AISC *Specification*. In addition to the detailed provisions, the *Specification* contains User Notes and a detailed Commentary that provides insights into the source and application of the provisions. The reader interested in additional background on the provisions discussed in this book is encouraged to investigate the materials cited in the appropriate sections of the Commentary. The *Specification* contains 14 chapters and 8 appendices. To provide a concise guide to the use of the *Specification*, a brief description is given here.

Chapter A: General Provision. This chapter provides the scope of the *Specification* and summarizes all referenced specifications, codes, and standards. It also provides the requirements for materials to be used in structural steel design and the design documents necessary to communicate that design.

Chapter B: Design Requirements. This chapter gives the general requirements for analysis and design that are applicable throughout the entire *Specification*. It provides the charging language needed for application of the subsequent chapters.

Chapter C: Design for Stability. This chapter, along with Appendix 7, addresses the requirements for the design of structures to ensure stability. It details those factors that must be taken into consideration in any analysis and design.

Chapter D: Design of Members for Tension. This chapter applies to the design of members subjected to axial tension resulting from forces acting through the centroidal axis.

Chapter E: Design of Members for Compression. This chapter addresses members subjected to axial compression resulting from forces applied at the centroidal axis.

Chapter F: Design of Members for Flexure. This chapter applies to members loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting. This is referred to as simple bending about one axis.

Chapter G: Design of Members for Shear. This chapter addresses webs of singly or doubly symmetric members subject to shear in the plane of the web. It also addresses other shapes such as single angles and hollow structural sections.

Chapter H: Design of Members for Combined Forces and Torsion. This chapter addresses design of members subject to an axial force in combination with flexure about one or both axes, with or without torsion. It also applies to members subjected to torsion only.

Chapter I: Design of Composite Members. This chapter addresses the design of members composed of steel shapes and concrete working together as a member. It addresses compression, flexure, and combined forces.

Chapter J: Design of Connections. This chapter addresses the design of connections, including the connecting elements, the connectors, and the connected portions of members.

Chapter K: Additional Requirements for HSS and Box Section Connections. This chapter addresses requirements in addition to those given in Chapter J for the design of connections to hollow structural sections and built-up box sections of uniform thickness and connections between HSS and box members.

Chapter L: Design for Serviceability. This chapter summarizes the performance requirements for the design of a serviceable structure.

Chapter M: Fabrication and Erection. This chapter addresses the requirements for shop drawings, fabrication, shop painting, and erection.

Chapter N: Quality Control and Quality Assurance. This chapter addresses the requirements for ensuring quality of the constructed project.

Appendix 1: Design by Advanced Analysis. The body of the *Specification* addresses design based on an elastic analysis. This appendix addresses design by alternative methods generally referred to as advanced methods. It includes the classical plastic design method and design by direct modeling of imperfections.

Appendix 2: Design for Ponding. This appendix provides methods for determining whether a roof system has sufficient strength and stiffness to resist the influence of water collecting on the surface and forming a pond.

Appendix 3: Fatigue. This appendix provides requirements for addressing the influence of high cycle loading on members and connections that could lead to cracking and progressive failure. For most building structures, fatigue is not an issue of concern.

Appendix 4: Structural Design for Fire Conditions. This appendix provides the criteria for evaluation of structural steel subjected to fire conditions, including (1) the prescriptive approach provided for in the model building code and most commonly used in current practice and (2) the engineered approach.

Appendix 5: Evaluation of Existing Structures. This appendix provides guidance on the determination of the strength and stiffness of existing structures by load tests or a combination of tests and analysis.

Appendix 6: Member Stability Bracing. This appendix details the criteria for ensuring that column, beam and beam-column bracing has sufficient strength and stiffness to meet the requirements for member bracing assumed in the provisions of the *Specification* for design of those members.

Appendix 7: Alternative Methods of Design for Stability. This appendix, along with Chapter C, provides methods of designing structures to ensure stability. Two alternative methods are provided here, including the method most commonly used in past practice.

Appendix 8: Approximate Second-Order Analysis. This appendix provides a method for obtaining second-order effects by an amplified first-order analysis. The provisions are limited to structures supporting load primarily through vertical columns.

Each chapter of this book will identify those chapters of the *Specification* that are pertinent to that chapter. The reader is encouraged to become familiar with the organization of the *Specification*.

1.3 THE MANUAL

The *AISC Steel Construction Manual*, 15th edition, is the latest in a series of manuals published to assist the building industry in designing safe and economical steel building structures. The first edition was published in 1928 and the ninth edition in 1989. These manuals addressed design by the allowable stress method. In 1986 the first edition of the load and resistance factor design method manual was published, with the third edition published in 1999. The next in this unbroken string of manuals published in support of steel design and construction was the first manual to unify these two design methods and was published in 2005 as the 13th edition. The current edition of the *Manual* is the 15th. Students who purchase the *Manual* through the AISC Student Discount Program also have an opportunity to apply for a free AISC Student Membership at the same time. Students are encouraged to become AISC Student Members in order to take full advantage of all free member benefits.

As is the case for the *Specification*, AISC has two resources to assist in addressing the historic aspects of steel design and construction. The first is, again, AISC Design Guide 15, *AISC Rehabilitation and Retrofit Guide: A Reference for Historic Shapes and Specifications*. This Guide provides properties of beam and column sections as old as the wrought iron shapes produced as early as 1873. The second resource is the electronic *AISC Shapes Database*. This database is available through the AISC web site www.aisc.org. It is a searchable database with properties for all shapes produced since 1873, consistent with the printed data in Design Guide 15. Access to the electronic shapes database is free to AISC members.

The *Manual* is presented in 17 parts as follows:

Part 1: Dimensions and Properties

Part 2: General Design Considerations

Part 3: Design of Flexural Members

Part 4: Design of Compression Members

Part 5: Design of Tension Members

Part 6: Design of Members Subject to Combined Forces

Part 7: Design Considerations for Bolts

Part 8: Design Considerations for Welds

Part 9: Design of Connecting Elements

Part 10: Design of Simple Shear Connections

Part 11: Design of Partially Restrained Moment Connections

Part 12: Design of Fully Restrained Moment Connections

Part 13: Design of Bracing Connections and Truss Connections

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

Part 15: Design of Hanger Connections, Bracket Plates, and Crane-Rail Connections

Part 16: Specifications and Codes

Part 17: Miscellaneous Data and Mathematical Information

Each chapter of this book identifies those parts of the *Manual* that will be used with the material to be addressed. In many instances, the user will need to look in several parts of the *Manual* to fully understand the topics or solve the problems presented.

1.4 AISC WEB SITE RESOURCES

Another primary resource is the AISC web site, where there is information that is free to all visitors and additional electronic resources that are free to members only. Students will find a great deal of useful information on the AISC publications web site, www.aisc.org/epubs. The primary resources include electronic versions of the *Specification*, the Shapes Database, the Steel Construction Manual References, and the Steel Construction Manual Design Examples. The *Specification*, as described in Section 1.2 and the historic Shapes Database, as mentioned in Section 1.3, are available free to all through the web site. The 15th edition Steel Construction Manual Shapes Database is also available free to all. The AISC web site also includes an extensive array of journal and proceedings papers. All of the references cited in the Commentary and the *Manual*, for which AISC owns the copyright, are accessible under Steel Construction Manual Resources; Interactive Reference List.

Probably the most valuable aspect of the AISC web site for readers of this book is the complete set of the 15th edition Steel Construction Manual Design Examples. These examples are presented in four sections.

Section I: Examples Based on the AISC *Specification*. This section contains examples demonstrating the use of the specific provisions of the *Specification*, organized by *Specification* chapter.

Section II: Examples Based on the AISC *Steel Construction Manual*. This section contains examples of connection design using the *Specification* and the tables found in the *Manual*.

Section III: System Design Examples. This section contains examples associated with the design of a specific building and the application of the system-wide requirements.

Section IV: Additional Resources. This section provides design tables for higher-strength steels than published in the printed *Manual*.

Although the topics covered in this book are supported by calculated example problems, the reader might find the electronic Steel Construction Manual Design Examples helpful for further understanding of some of the specific provisions or design aids described in the book. In addition, some of the Design Examples go beyond the coverage in this book and provide additional useful information regarding typical design or detailing. The reader is encouraged to investigate what the AISC web site has to offer through both free and member only publications.

1.5 PRINCIPLES OF STRUCTURAL DESIGN

From the time an owner determines a need to build a building, through the development of conceptual and detailed plans, to completion and occupancy, a building project is a multi-faceted task that involves many professionals. The owner and the financial analysis team evaluate the basic economic criteria for the building. The architects and engineers form the design team and prepare the initial proposals for the building, demonstrating how the users' needs will be met. This teamwork continues through the final planning and design stages, where the design drawings, specifications, and contract documents are readied for the construction phase. During this process, input may also be provided by the individuals who will transform the plans into a real-life structure. The steel detailer, fabricator and erector all have a role in that process, and add their respective expertise to make the design constructible. Thus, those responsible for the construction phase of the project often help improve the design by taking into account the actual on-site requirements for efficient construction.

Once a project is completed and turned over to the owner, the work of the design teams is normally over. The operation and maintenance of the building, although major factors in the life of the structure, are not usually within the scope of the designer's responsibilities, except when significant changes in building use are anticipated. In such cases, a design team should verify that the proposed changes can be accommodated.

The basic goals of the design team can be summarized by the words *safety*, *function*, and *economy*. The building must be safe for its occupants and all others who may come in contact with it. It must neither fail locally nor overall, nor exhibit behavioral characteristics that test the confidence of rational human beings. To help achieve that level of safety, building codes and design specifications are published that outline the minimum criteria that any structure must meet.

The building must also serve its owner in the best possible way to ensure that the functional criteria are met. Although structural safety and integrity are of paramount importance, a building that does not serve its intended purpose will not have met the goals of the owner.

Last, but not least, the design, construction, and long-term use of the building should be economical. The degree of financial success of any structure will depend on a wide range of factors. Some are established prior to the work of the design team, whereas others are determined after the building is in operation. Nevertheless, the final design should, within all reasonable constraints, produce the lowest combined short- and long-term expenditures.

The AISC *Specification* follows the same principles. The mission of the AISC Committee on Specifications is to "develop the practice-oriented specification for structural steel buildings that provide for life safety, economical building systems, predictable behavior and response, and efficient use." Thus, this book emphasizes the practical orientation of this *Specification*.

1.6 PARTS OF THE STEEL STRUCTURE

All structures incorporate some or all of the following basic types of structural components:

1. Tension members
2. Compression members
3. Bending members
4. Combined force members
5. Connections

The first four items represent structural members. The fifth, connections, represents the contact regions between the structural members, which ensure that all components work together as a structure.

Detailed evaluations of the strength, behavior, and design criteria for these members are presented in the following chapters:

Tension members:	Chapter 4
Compression members:	Chapter 5

Bending members:	Chapters 6 and 7
Combined force members:	Chapter 8
Connections:	Chapters 10, 11, and 12

The strength and behavior of structural frames composed of a combination of these elements are covered in Chapters 8 and 13, and the special considerations that apply to composite (steel and concrete working together) construction are presented in Chapter 9. An introduction to the design of steel structures for earthquake loading is presented in Chapter 13. The properties of structural steel and the various shapes commonly used are discussed in Chapter 3, and a brief discussion of the types of loads and load combinations is presented in Chapter 2.

Tension members are typically found as web and chord members in trusses and open-web steel joists; as diagonal members in structural bracing systems; and as hangers for balconies, mezzanine floors, and pedestrian walkways. They are also used as sag rods for purlins and girts in many building types, as well as to support platforms for mechanical equipment and pipelines. Figures 1.1 and 1.2 illustrate typical applications of tension members in actual structures.

In the idealized case, tension members transmit concentric tensile forces only. In certain structures, reversals of the overall load may change the tension member force from tension to compression. Some members will be designed for this action; others will have been designed with the assumption that they will carry tension only.

The idealized tension member is analyzed with the assumption that its end connections are pins, which prevent any moment or shear force from being transmitted to the member. However, in an actual structure, the type of connection normally dictates that some bending may be introduced to the tension member. This is also the case when the tension member is directly exposed to some form of transverse load. Moments will also be introduced if the element is not perfectly straight, or if the axial load is not applied along the centroidal axis of the member.

The primary load effect in the tension member is a concentric axial force, with bending and shear considered as secondary effects.

Compression members are also known as columns, struts, or posts. They are used as web and chord members in trusses and joists and as vertical members in all types of building structures. Figure 1.3 shows a typical use of structural compression members.

Figure 1.1 Use of Tension Members in a Truss
Photo courtesy Ruby + Associates

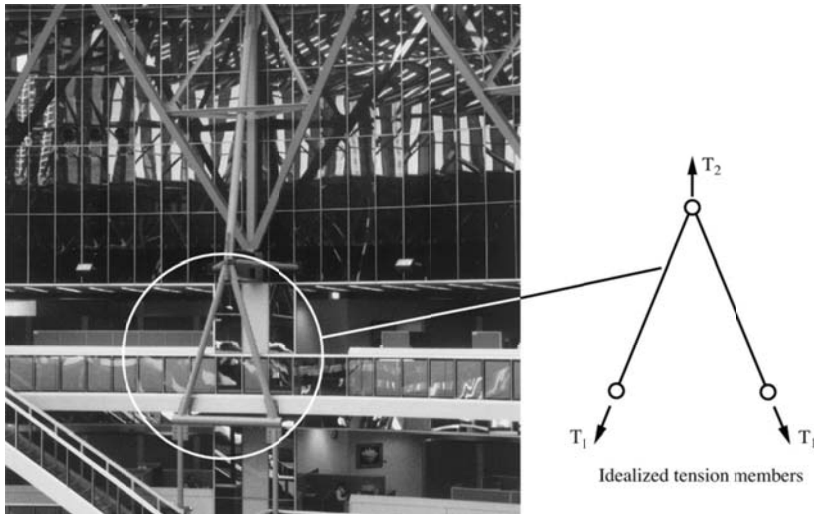


Figure 1.2 Use of Tension Members as Hangers

The idealized compression member carries only a concentric, compressive force. Its strength is heavily influenced by the distance between the supports, as well as by the support conditions. The basic column is therefore defined as an axially loaded member with pinned ends. Historically, design rules for compression members have been based on the behavior and strength of this idealized compression member.

The basic column is practically nonexistent in real structures. Realistic end supports rarely resemble perfect pins; the axial load is normally not concentric, due to the way the surrounding structure transmits its load to the member; and beams and similar components are likely to be connected to the column in such a way that moments are introduced. All of these conditions produce bending effects in the member, making it a combined force member or beam-column, as distinct from the idealized column.

The primary load effect in the pinned-end column is therefore a concentric axial compressive force accompanied by the secondary effects of bending and shear.

Bending members are known as beams, girders, joists, spandrels, purlins, lintels, and girts. Although all of these are bending members, each name implies a certain structural application within a building:

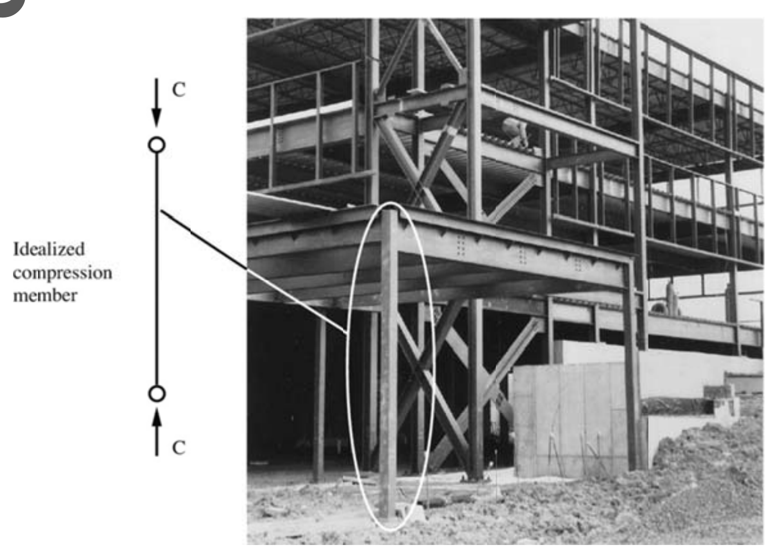


Figure 1.3 Use of Columns in a Building Frame

1. Beams, girders, and joists form part of common floor systems. The beams are most often considered as the members that are directly supported by girders, which in turn are usually supported by columns. Joists are beams with fairly close spacing. A girder may generally be considered a higher-order bending member compared with a beam or joist. However, variations to this basic scheme are common.

2. The bending members that form the perimeter of a floor or roof plan in a building are known as spandrels or spandrel beams. Their design may be different from other beams and girders because the load comes primarily from one side of the member.

3. Bending members in roof systems that span between other bending members are usually referred to as purlins.

4. Lintels are bending members that span across the top of openings in walls, usually carrying the weight of the wall above the opening as well as any other load brought into that area. They typically are seen spanning across the openings for doors and windows.

5. Girts are used in exterior wall systems. They transfer the lateral load from the wall surface to the exterior columns. They may also assist in supporting the weight of the wall.

Figure 1.4 shows beams and girders in an actual structure under construction. The idealized beam is shown in the figure as a member with a uniform load supported on simple supports.

The basic bending member carries transverse loads that act in a plane containing the longitudinal centroidal axis of the member. The primary load effects are bending moment and shear force. Axial forces and torsion may occur as secondary effects.

The most common *combined force member* is known as a beam-column, implying that this structural element is simultaneously subjected to bending and axial compression. Although the presence of both bending and axial tension represents a potential loading case for the combined force member, this case is not as critical or common as the beam-column loading case.

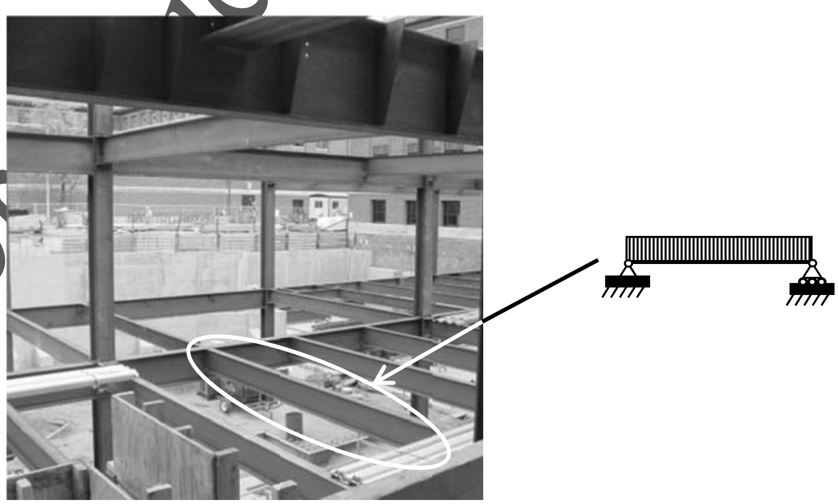


Figure 1.4 Building Structure Showing Bending Members

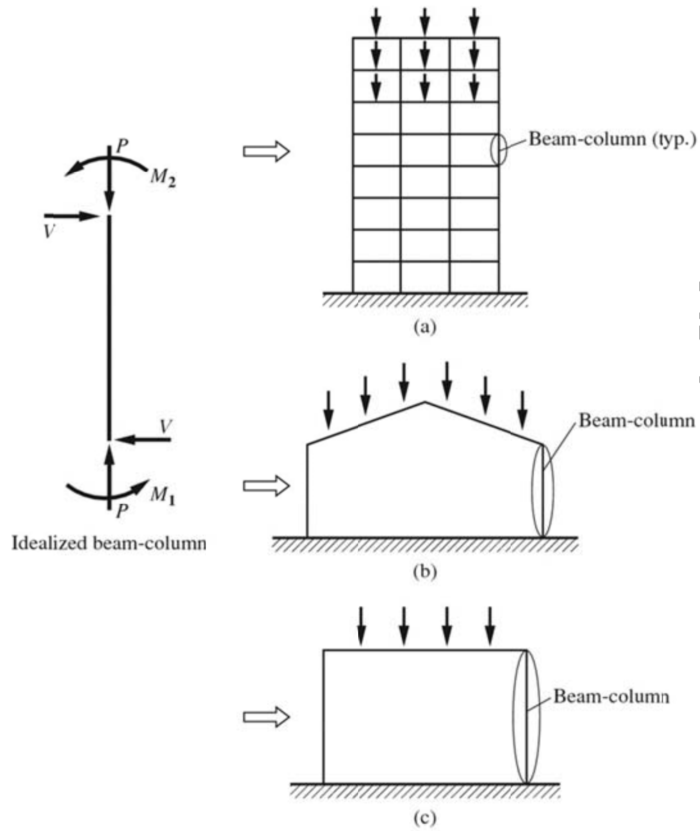


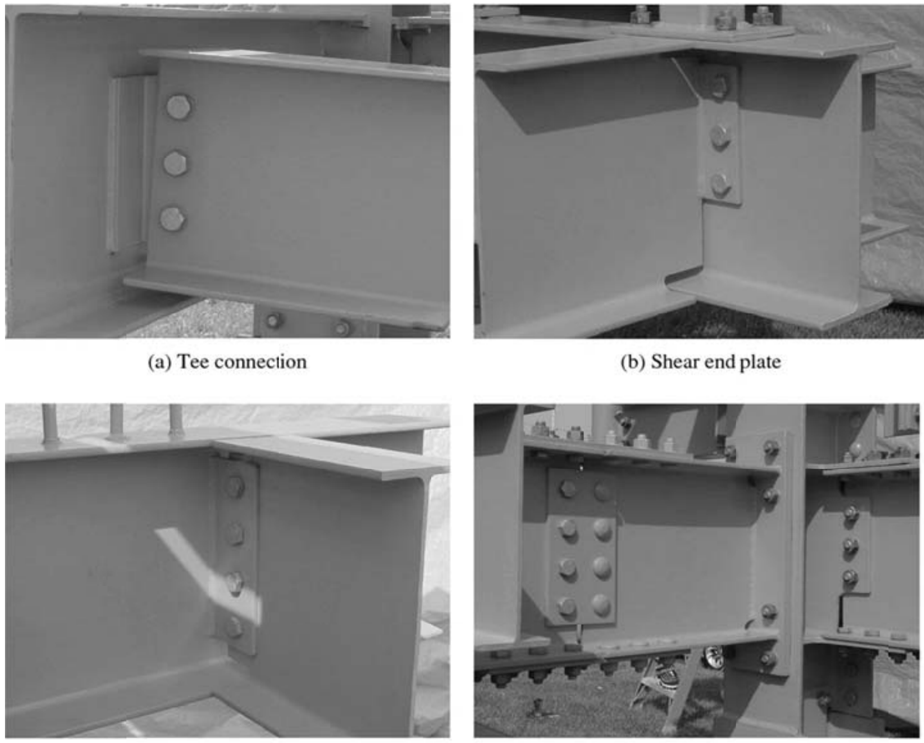
Figure 1.5 Schematic Representation of Steel Frames in Which the Vertical Members Are Subjected to Axial Loads and Bending Moments

Figure 1.5a is a schematic illustration of a multi-story steel frame where the beams and columns are joined with rigid connections. Because of the geometric configuration, the types of connections, and the loading pattern, the vertical members are subjected to axial loads and bending moments. This is a typical application of practical beam-columns; other examples are the members of the gable frame shown in Figure 1.5b and the vertical components of a single-story portal frame shown in Figure 1.5c.

The beam-column may be regarded as the general structural element, where axial forces, shear forces, and bending moments act simultaneously. Thus, the basic column may be thought of as a special case, representing a beam-column with no moments or transverse loads. Similarly, the basic bending member may be thought of as a beam-column with no axial load. Therefore, the considerations that must be accounted for in the design of both columns and beams must also be applied to beam-columns.

Because of the generalized nature of the combined force element, all load effects are considered primary. However, when the ratio of axial load to axial load strength in a beam-column becomes high, column behavior will overshadow other influences. Similarly, when the ratio of applied moment to moment strength is high, beam behavior will outweigh other effects. The beam-column is an element in which a variety of different force types interact. Thus, practical design approaches are normally based on interaction equations.

Connections are the collection of elements that join the members of a steel structure together. Whether they connect the axially loaded members in a truss or the beams and columns



(a) Tee connection (b) Shear end plate

(c) Shear tab (d) Moment end plate

Figure 1.6 Building Connections

of a multi-story frame, connections must ensure that the structural members function together as a unit, consistent with the assumptions made in the design.

The fasteners used in structural steel connections today are almost entirely limited to bolts and welds. The load effects that the various elements of the connection must resist are a function of the specific connection type being considered. They include all of the possible forces and moments. Figure 1.6 illustrates a variety of connections. The idealized representations for connections are presented in Chapters 10, 11, and 12.

1.7 TYPES OF STEEL STRUCTURES

It is difficult to classify steel structures into neat categories, due to the wide variety of systems available to the designer. The elements of the structure, as defined in Section 1.6, are combined to form the total structure of a building, which must safely and economically carry all imposed loads. This combination of members is usually referred to as the framing system.

Steel-framed buildings come in a wide variety of shapes and sizes and in combinations with other structural materials. A few examples are given in the following paragraphs, to set the stage for the application of structural design presented in subsequent chapters.

1.7.1 Bearing Wall Construction

Bearing wall construction is primarily used for one- or two-story buildings, such as storage warehouses, shopping centers, office buildings, and schools. This system normally uses brick or concrete block masonry walls, on which are placed the ends of the flexural members supporting the floor or roof. The flexural members are usually hot-rolled structural steel shapes, alone or in

Figure 1.7 Bearing Wall Building
Photo courtesy Douglas Steel Fabricating Corporation

combination with open web steel joists or cold-formed steel shapes. An example of a bearing wall building is shown in Figure 1.7.

1.7.2 Beam-and-Column Construction

Beam-and-column construction is the most commonly used system for steel structures today. It is suitable for large-area buildings such as schools and shopping centers, which often have no more than two stories but may have a large number of spans. It is also suitable for buildings with many stories. Columns are placed according to a regular, repetitious grid that supports the beams, girders, and joists, which are used for the floor and roof systems. The regularity of the floor plan lends itself to economy in fabrication and erection, because most of the members will be of the same size. An example of this type of structure is shown in Figure 1.8

For multi-story buildings, the use of composite steel and concrete flexural members affords additional savings. Further advances can be expected as designers become more familiar with the use of composite columns and other elements of mixed construction systems.

Figure 1.8 Beam-and-Column Building
Photo courtesy Douglas Steel Fabricating Corporation

Figure 1.9 Braced Beam-and Column Building
Photo courtesy Douglas Steel Fabricating Corporation

Beam-and-column structures rely on either their connections or a separate bracing system to resist lateral loads. A frame in which all connections are moment resistant provides resistance against the action of lateral loads, such as wind and earthquakes, and overall structural stability, through the bending stiffness of the overall frame.

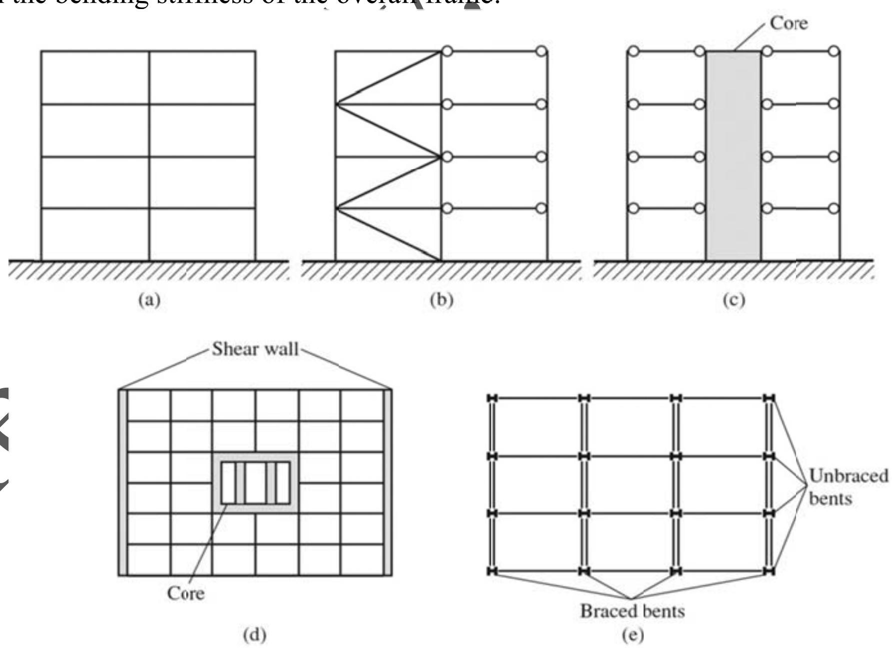


Figure 1.10 Idealized Illustration of Several Types of Beam-and-Column Framed Structures(a) moment-resistant frame; (b) truss-braced frame; (c) core-braced frame; (d) floor plan of shear wall and core-braced building; (e) floor plan of building with a combination of braced and unbraced bents.

A frame without member-end restraint needs a separate lateral load resisting system, which is often afforded by having the elements along one or more of the column lines act as braced frames, as seen in Figure 1.9. One of the most common types of bracing is the vertical truss, which is designed to take the loads imposed by wind and seismic action. Other bracing schemes involve shear walls and reinforced concrete cores. The latter type may also be referred to as a braced core system and can be highly efficient because of the rigidity of the box-shaped cross section of the core. The core serves a dual purpose in this case: In addition to providing the bracing system for the building, it serves as the vertical conduit in the completed structure for all of the necessary services, including elevators, staircases, electricity, and other utilities.

Combinations of these types of construction are also common. For example, frames may have been designed as moment resistant in one direction of the building and as truss braced in the other. Of course, such a choice recognizes the three-dimensional nature of the structure.

Figure 1.10 shows an idealized representation of several types of beam-and-column framed structures.

1.7.3 Long-Span Construction

This type of construction encompasses steel-framed structures with long spans between the vertical load-carrying elements, such as covered arenas. The long distances may be spanned by one-way trusses, two-way space trusses, or plate and box girders. A long-span structure is shown in Figure 1.11. Arches or cables could also be used, although they are not considered here.

Long-span construction is also used in buildings that require large, column-free interiors. In such cases the building may be a core- or otherwise braced structure, where the long span is the distance from the exterior wall to the core.

Many designers would also characterize single-story rigid frames as examples of long-span construction systems. Depending on the geometry of the frame, such structures can span substantial distances, often with excellent economy.

Figure 1.11 Long-Span Structure
Photo courtesy Douglas Steel Fabricating Corporation

Figure 1.12 High-Rise Building Structure
Photo courtesy Douglas Steel Fabricating Corporation

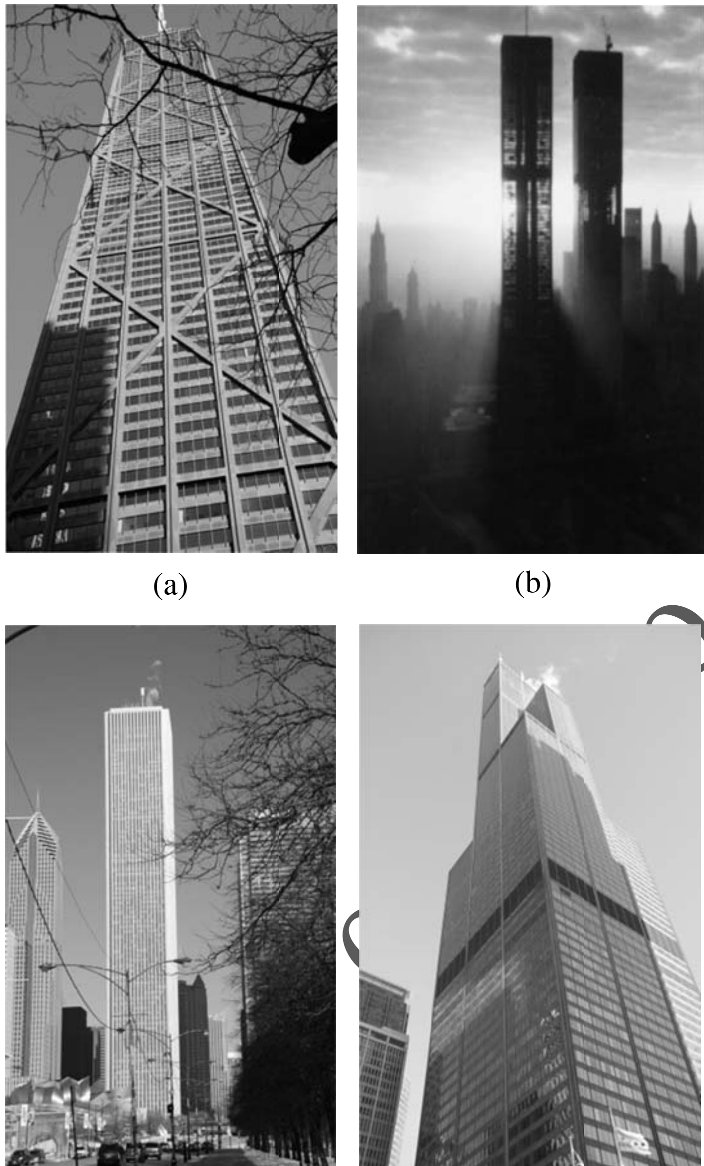
1.7.4 High-Rise Construction

High-rise construction refers to multi-story buildings of significant height; an example is shown in Figure 1.12. The large heights and unique problems encountered in the design of such structures warrant treating them independently from typical beam-and-column construction. In addition, over the past 40 years several designers have developed a number of new concepts in multi-story frame design, such as the super composite column and the steel plate shear wall.

Particular care must be exercised in the choice and design of the lateral load resisting system in high-rise construction. It is not just a matter of extrapolating from the principles used in the analysis of lower-rise structures, because many effects play a major role in the design of high-rise buildings but have significantly less impact on frames of smaller height. These effects are crucial to the proper design of the high-rise structure.

Some of these effects may be referred to as second-order effects, because they cannot be quantified through a normal, linearly elastic analysis of the frame. Although second-order effects are present in all structures, they may be more significant in high-rise structures. For example, when a structure is displaced laterally, additional moment is induced in a column due to the eccentricity of the column loads. When added to the moments and shears produced by gravity and wind loads, the resulting effects may be significantly larger than those computed without considering the second-order effects. A designer who does not incorporate both types of effects will be making a serious and perhaps unconservative error.

Framing systems for high-rise buildings reflect the increased importance of lateral load resistance. Thus, attempts at making the perimeter of a building act as a unit or tube have proven quite successful. This tube may be in the form of a truss, as with the John Hancock Building in Chicago, Illinois, shown in Figure 1.13a, or a frame, as in the former World Trade Center in New York City, shown in Figure 1.13b; a solid wall tube with cutouts for windows, as used in the Aon Center in Chicago, shown in Figure 1.13c; or several interconnected or bundled tubes, such as in the Willis Tower in Chicago (formerly known as the Sears Tower), shown in Figure 1.13d.



(a) (b) (c) (d)

Figure 1.13 High-Rise Buildings (a) the John Hancock Center; (b) the World Trade Center; (c) the Aon Center; (d) the Willis Tower.
Photo (b) courtesy Leslie E. Robertson Associates, RLLP

1.7.5 Gable-Frame Construction

Many designers include the single-story frame as part of the long-span construction category. These structures lend themselves particularly well to fully welded construction. The pre-engineered building industry has capitalized on the use of this system through fine-tuned designs of frames for storage warehouses, industrial buildings, temporary and permanent office buildings, and similar types of structures. An example of a pre-engineered metal building with a gable frame is seen in Figure 1.14.

Figure 1.14 Pre-engineered Metal Building
Photo courtesy Metal Building Manufacturers Association

1.8 DESIGN PHILOSOPHIES

A successful structural design results in a structure that is safe for its occupants, can carry the design loads without overstressing any components, does not deform or vibrate excessively, and is economical to build and operate for its intended life span. Although economy may appear to be the primary concern of an owner, safety must be the primary concern of the engineer. Costs of labor and materials will vary from one geographic location to another, making it almost impossible to design a structure that is equally economical in all locations. Because the foremost task of the designer is to produce a safe and serviceable structure, design criteria such as those published by the American Institute of Steel Construction are based on technical models and considerations that predict structural behavior and material response. The use of these provisions by the designer will dictate the economy of a particular solution in a particular location and business climate.

To perform a structural design, it is necessary to quantify the causes and effects of the loads that will be exerted on each element throughout the life of the structure. This is generally termed the *load effect* or the *required strength*. It is also necessary to account for the behavior of the material and the shapes that compose these elements. This is referred to as the *nominal strength* or *capacity* of the element.

In its simplest form, structural design is the determination of member sizes and their corresponding connections, so that the strength of the structure is greater than the load effect. The degree to which this is accomplished is often termed the *margin of safety*. Numerous approaches for accomplishing this goal have been used over the years.

Although past experience might seem to indicate that the structural designer knows the exact magnitude of the loads that will be applied to the structure, and the exact strength of all of the structural elements, this is usually not the case. Design loads are provided by many codes and standards and, although the values given are specific, significant uncertainty is associated with those magnitudes. Loads, load factors, and load combinations are discussed in Chapter 2.

As is the case for loading, significant uncertainty is associated with the determination of the behavior and strength of structural members. The true indication of load-carrying capacity is given by the magnitude of the load that causes the failure of a component or the structure as a whole. Failure may either occur as the physical collapse of part of the building, or be considered to have occurred if deflections, for instance, exceed certain predetermined values. Whether the failure is the result of a lack of strength (collapse) or stiffness (deflection), these phenomena reflect the limits of acceptable behavior of the structure. Based on these criteria, the structure is said to have reached a specific *limit state*. A strength failure is termed an *ultimate limit state,* whereas a failure to meet operational requirements, such as deflection, is termed a *serviceability limit state*.

Regardless of the approach to the design problem, the goal of the designer is to ensure that the load on the structure and its resulting load effect, such as bending moment, shear force, and axial force, in all cases are sufficiently below each of the applicable limit states. This ensures that the structure meets the required level of safety or reliability.

Three approaches to the design of steel structures are permitted by the AISC *Specification*:

1. Allowable strength design (ASD)
2. Load and resistance factor design (LRFD)
3. Design by inelastic analysis

The design approaches represent alternative ways of formulating the same problem, and all have the same goal. All three are based on the nominal strength of the element or structure. The nominal strength, most generally expressed as R_n, is determined in exactly the same way, from the exact same equations, whether used in ASD or LRFD. Some formulations of design by inelastic analysis, such as plastic design, also use these same nominal strength equations whereas other approaches to inelastic design model in detail every aspect of the structural behavior and do not rely on the equations provided through the *Specification*. The use of a single nominal strength expression for both ASD and LRFD permits the unification of these two design approaches. It will become clear throughout this book how this approach has simplified steel design for those who have struggled in the past with comparing the two available philosophies. The following sections describe these three design approaches, any one of which is an acceptable approach to structural steel design according to the AISC *Specification*.

1.9 FUNDAMENTALS OF ALLOWABLE STRENGTH DESIGN (ASD)

Prior to 2005, allowable strength design was referred to as allowable stress design. It is the oldest approach to structural design in use today and has been the foundation of AISC Specifications since the original provisions of 1923. Allowable stress design was based on the assumption that under actual load, stresses in all members and elements would remain elastic. To meet this requirement, a safety factor was established for each potential stress-producing state. Although historically ASD was thought of as a stress-based design approach, the allowable strength was always obtained by using the proper combination of the allowable stress and the corresponding section property, such as area or elastic section modulus.

The current allowable strength design approach is based on the concept that the required strength of a component is not to exceed a certain permitted or allowable strength under normal in-service conditions. The required strength is determined on the basis of specific ASD load combinations and an elastic analysis of the structure. The allowable strength incorporates a factor of safety, Ω, and uses the nominal strength of the element under consideration. This strength could be presented in the form of a stress if the appropriate section property is used. As a result of doing this, the resulting stresses will most likely again be within the elastic range, although this is not a preset requirement of the *Specification*.

The magnitude of the factor of safety and the resulting allowable strength depend on the particular governing limit state against which the design must produce a certain margin of safety.

Safety factors are obtained from the *Specification*. This requirement for ASD is provided in Section B3.2 of the *Specification* as

$$R_a \le \frac{R_n}{\Omega} \qquad \text{(AISC B3-2)}$$

which can be stated as

$$\text{Required Strength (ASD)} \le \frac{\text{Nominal Strength}}{\text{Safety Factor}} = \text{Allowable Strength}$$

The governing strength depends on the type of structural element and the limit states being considered. Any single element can have multiple limit states that must be assessed. The safety factor specified for each limit state is a function of material behavior and the limit state being considered. Thus, it is possible for each limit state to have its own unique safety factor. For example, the limit state of yielding of a tension member is given by

$$P_n = F_y A_g$$

where F_y is the steel yield strength and A_g is the gross area of the member. The safety factor is $\Omega = 1.67$. Thus, for steel with a yield strength of 50 ksi, the allowable strength is

$$\frac{P_n}{\Omega} = \frac{50.0A_g}{1.67} = 30A_g$$

Design by ASD requires that the allowable stress load combinations of the building code be used. Loads and load combinations are discussed in detail in Chapter 2.

1.10 FUNDAMENTALS OF LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

Load and resistance factor design explicitly incorporates the effects of the random variability of both strength and load. Because the method includes the effects of these random variations and formulates the safety criteria on that basis, it is expected that a more uniform level of reliability, and thus safety, for the structure and all of its components will be attained.

LRFD is based on the concept that the required strength of a component under LRFD load combinations is not to exceed the design strength. The required strength is obtained by increasing the load magnitude by load factors that account for load variability and load combinations. The design strength is obtained by reducing the nominal strength by a resistance factor that accounts for the many variables that impact the determination of member strength. Load factors for LRFD are obtained from the building codes for strength design and will be discussed in Chapter 2. As for ASD safety factors, the resistance factors are obtained from the *Specification*.

The basic LRFD provision is provided in Section B3.1 of the *Specification* as

$$R_u \le \phi R_n \qquad \text{(AISC B3-1)}$$

which can be stated as

$$\text{Required Strength (LRFD)} \le \text{Resistance Factor} \times \text{Nominal Strength} = \text{Design Strength}$$

Again considering the limit state of yielding of a tension member,

$$P_n = F_y A_g$$

and the resistance factor is $\phi = 0.90$. For steel with a yield strength of 50 ksi, the design strength is

$$\phi P_n = 0.90(50)A_g = 45A_g$$

LRFD has been a part of the AISC Specifications since it was first issued in 1986.

1.11 INELASTIC DESIGN

The *Specification* permits a wide variety of formulations for the inelastic analysis of steel structures through the use of Appendix 1. Any inelastic analysis method will require that the structure and its elements be modeled in sufficient detail to account for all types of behavior. An analysis of this type must be able to track the structure's behavior from the unloaded condition through every load increment to complete structural failure. The only inelastic design approach that will be discussed in this book is plastic design (PD).

Plastic design is an approach that has been available as an optional method for steel design since 1961, when it was introduced as Part 2 of the then current *Specification*. The limiting condition for the structure and its members is attainment of the load that would cause the structure to collapse, usually called the ultimate strength or the plastic collapse load. For an individual structural member this means that its plastic moment capacity has been reached. In most cases, due to the ductility of the material and the member, the ultimate strength of the entire structure will not have been reached at this stage. The less stressed members can take additional load until a sufficient number of members have exhausted their individual capacities so that no further redistribution or load sharing is possible. At the point where the structure can take no additional load, the structure is said to have collapsed. This load magnitude is called the *collapse load* and is associated with a particular *collapse mechanism*.

The collapse load for plastic design is the service load times a certain load factor. The limit state for a structure that is designed according to the principles of plastic design is therefore the attainment of a mechanism. For this to occur, all of the structural members must be able to develop the yield stress in all fibers at the most highly loaded locations.

There is a fine line of distinction between the load factor of PD and the safety factor of ASD. The former is the ratio between the plastic collapse load and the service or specified load for the structure as a whole, whereas the latter is an empirically developed, experience-based term that represents the relationship between the elastic strength of the elements of the structure and the various limiting conditions for those components. Although numerically close, the load factor of plastic design and the factor of safety of allowable stress design are not the same parameter.

1.12 STRUCTURAL SAFETY AND INTEGRITY

The preceding discussions of design philosophies indicate that although the basic goal of any design process is to ensure that the end product is a safe and reliable structure, the ways in which this is achieved may vary substantially.

In the past, the primary goal for safety was to provide an adequate margin against the consequences of overload. Load factor design and its offshoots were developed to take these considerations into account. In real life, however, many other factors also play a role. These include, but are not limited to the following:

1. Variations of material strength

2. Variations of cross-sectional size and shape

3. Accuracy of method of analysis

4. Influence of workmanship in shop and field

5. Presence and variation of residual stresses

6. Lack of member straightness

7. Variations of locations of load application points

These factors consider only some of the sources of variation of the strength of a structure and its components. An even greater source of variation is the loading, which is further complicated by the fact that different types of load have different variational characteristics.

Thus, a method of design that does not attempt to incorporate the effects of strength and load variability will be burdened with unaccounted-for sources of uncertainty. The realistic solution, therefore, is to deal with safety as a probabilistic concept. This is the foundation of load and resistance factor design, where the probabilistic characteristics of load and strength are evaluated and the resulting safety margins determined statistically. Each load type is given its own specific factor in each combination, and each material limit state is also given its own factor. This method recognizes that there is always a finite, though very small, chance that structural failure will actually occur. However, this method does not attempt to attach a specific value to this probability. No specific level of probability of failure is given or implied by the *Specification*.

In ASD, the variabilities of load and strength are not treated explicitly as separate parameters. They are lumped together through the use of a single factor of safety. The factor of safety varies with each strength limit state but does not vary with load source. ASD can be thought of as LRFD with a single load factor. LRFD designs are generally expected to have a more uniform level of reliability than ASD designs. That is, the probability of failure of each element in an LRFD design will be the same, regardless of the type of load or load combination. However, a detailed analysis of reliability under the LRFD provisions shows that reliability still varies under different load combinations. In ASD there is no attempt to attain uniform reliability; rather, the goal is to simply have a safe structure, although some elements will be safer than others.

For the development of LRFD, load effect (member force), Q, and resistance (strength), R, are assumed to each have a variability that can be described by the normal distributions shown by the bell-shaped curves in Figure 1.15. Structures can be considered safe as long as the resistance is greater than the load effect, $R > Q$. If it were appropriate to concentrate solely on the mean values, Q_m and R_m, it would be relatively easy to ensure a structure's safety. However, the full representation of the data shows an area where the two curves overlap.

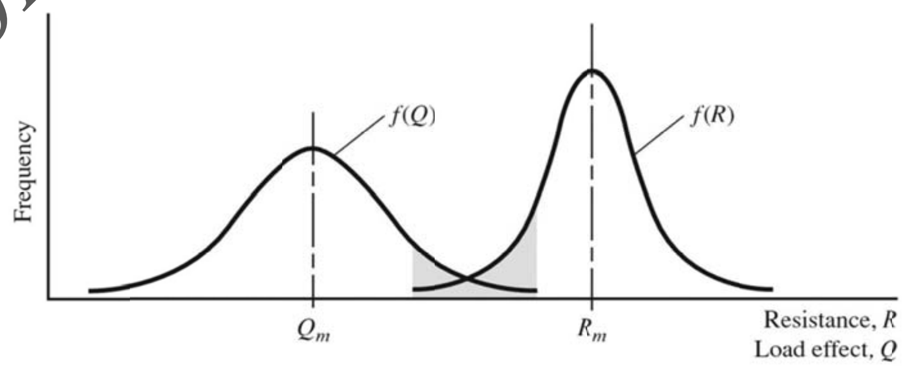


Figure 1.15 Probability Distribution, R and Q

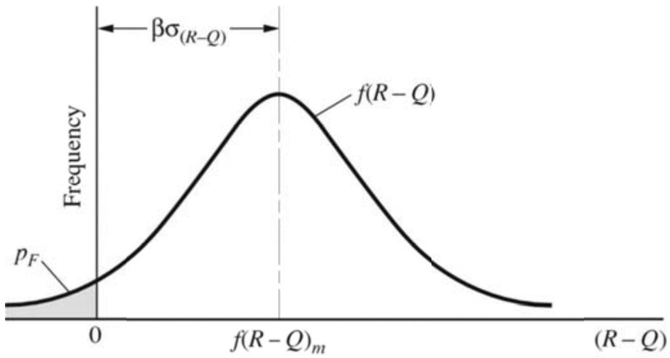


Figure 1.16 Probability Distribution, $(R-Q)$

This area represents cases where the load effect exceeds the resistance and therefore identifies occurrences of failure. Safety of the structure is a function of the size of this region of overlap. The smaller the region of overlap is, the lower the probability of failure.

Another approach to presenting the data is to look at the difference between resistance and load effect. Figure 1.16 shows the same data as Figure 1.15 but presents it as $(R-Q)$. For all cases where $(R-Q) < 0$, the structure is said to have failed, and for all cases where this difference is positive, the structure is considered safe. In this presentation of the data, the shaded area to the left of the origin represents the probability of failure. To limit that probability of failure, the mean value, $(R-Q)_m$, must be maintained at an appropriate distance from the origin. This distance is shown in Figure 1.16 as $\beta\sigma_{(R-Q)}$, where β is the reliability index and $\sigma_{(R-Q)}$ is the standard deviation of $(R-Q)$.

A third representation of the data is shown in Figure 1.17. In this case, the data is presented as $\ln(R/Q)$. The logarithmic form of the data is a well-conditioned representation and is more useful in the derivation of the factors required in LRFD. If we know the exact distribution of the resistance and load effect data, the probability of failure can be directly related to the reliability index β. Unfortunately, we know the actual distributions for relatively few resistance and load effect components. Thus, we must rely on other characteristics of the data, such as means and standard deviations.

The statistical analyses required to establish an appropriate level of reliability have been carried out by the appropriate specification committees, and the resulting load factors, resistance factors, and safety factors have been established. Load factors are presented in the building codes whereas resistance factors and safety factors for each limit state are given in the *Specification*.

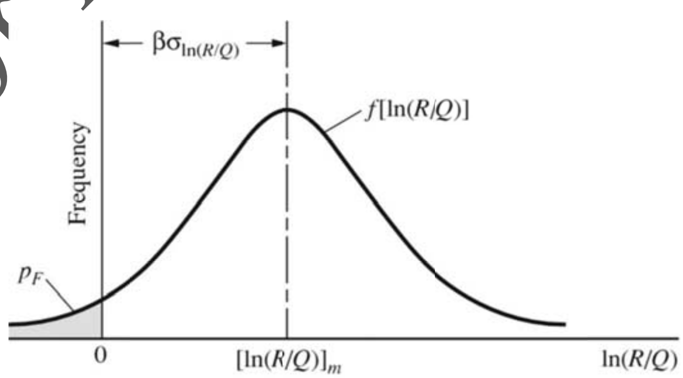


Figure 1.17 Probability Distribution, $\ln(R/Q)$

Since the load combinations and resistance and safety factors have been established, the reliability can be determined for specific design situations. The reliability index, β, is given in the *Specification* Commentary as

$$\beta = \frac{\ln\left(R_m/Q_m\right)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(AISC C-B3-2)}$$

where R_m is the mean resistance, Q_m is the mean load effect, as discussed earlier, and V_R and V_Q are the coefficients of variation of resistance and load effect respectively. Design according to LRFD is given by

$$R_u \le \phi R_n \qquad \text{(AISC B3-1)}$$

where the required strength, R_u is another term for the load effect, Q, R_n is the nominal strength and ϕ is the resistance factor. The reliability of design is determined when the required strength is exactly equal to the available strength. Thus, Equation B3-1 can be rewritten as $Q = \phi R_n$. The load effect will depend on the load combination being considered. Thus, for the LRFD live load plus dead load combination, written in terms of the live-to-dead load ratio, L/D,

$$Q = 1.2D + 1.6L = \left(1.2 + 1.6\left(L/D\right)\right)D = \phi R_n \qquad (1)$$

From Ravindra and Galambos[1] the mean resistance is given by

$$R_m = R_n M_m F_m P_m \qquad (2)$$

and the coefficient of variation of the resistance is given by

$$V_R = \sqrt{V_m^2 + V_F^2 + V_P^2} \qquad (3)$$

M_m is the mean of the ratio of the actual yield stress to the specified yield stress and V_M is the coefficient of variation; F_m is the mean of the ratio of the actual section property to the *Manual* value and V_F is the coefficient of variation; and P_m is the mean of the ratio of the test specimen strength to the predicted strength using the *Specification* equations and the actual material and geometric properties and V_P is the coefficient of variation.

Solving Equation 1 for R_n and substituting into Equation 2 yields

$$R_m = \frac{\left(1.2 + 1.6\left(L/D\right)\right)D}{\phi} M_m F_m P_m \qquad (4)$$

Rearranging Equation 2 yields

$$M_m F_m P_m = \frac{R_m}{R_n} \qquad (5)$$

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[1] Ravindra, M.K. and Galambos, T.V. (1978), "Load and Resistance Factor Design for Steel," *Journal of the Structural Division*, American Society of Civil Engineers, Vol. 104, No. ST9, September, pp. 1,337–1,353.

Thus, combining Equations 4 and 5 gives

$$R_m = D\left(1.2 + 1.6\left(L/D\right)\right)\left(\frac{R_m}{\phi R_n}\right) \tag{6}$$

From Ravindra and Galambos the mean load effect for dead load plus live load is

$$Q_m = D_m + L_m \tag{7}$$

which can, with some manipulation, be rewritten as

$$Q_m = D_m + L_m = \left(D_m/D + \left(L_m/L\right)\left(L/D\right)\right)D \tag{8}$$

They also give the coefficient of variation of the load effect which can be written as a function of the live-to-dead load ratio as

$$V_Q = \frac{\sqrt{\left(\left(D_m/D\right)V_D\right)^2 + \left(\left(L_m/L\right)\left(L/D\right)V_L\right)^2}}{Q_m} \tag{9}$$

If Equations 6 and 8 are substituted into Equation C-B3-2 the reliability index, β, will be given in terms of the live-to-dead load ratio, L/D, and the resistance factor, ϕ. Thus,

$$\beta = \frac{1}{\sqrt{V_R^2 + V_Q^2}} \ln\left[\frac{R_m}{\phi R_n}\left(\frac{1.2 + 1.6\left(L/D\right)}{\left(D_m/D\right) + \left(L_m/L\right)\left(L/D\right)}\right)\right] \tag{10}$$

For LRFD, the live plus dead load combinations are 1.4D and (1.2D + 1.6L). The effective dead load factor as a function of the live-to-dead-load ratio can be taken as

$$\gamma_{\text{LRFD}_i} = \max\begin{bmatrix} 1.4 \\ 1.2 + 1.6\left(L/D\right)_i \end{bmatrix} \tag{11}$$

and the mean load effect dead load multiplier as

$$Q_{m_i} = D_m/D + \left(L_m/L\right)\left(L/D\right)_i \tag{12}$$

Thus, Equation 10 can be generalized to address other LRFD load combinations as follows:

$$\beta = \frac{1}{\sqrt{V_R^2 + V_Q^2}} \ln\left[\frac{R_m}{\phi R_n}\left(\frac{1.2 + 1.6\left(L/D\right)}{\left(D_m/D\right) + \left(L_m/L\right)\left(L/D\right)}\right)\right] = \frac{1}{\sqrt{V_R^2 + V_{Q_i}^2}} \ln\left[\frac{R_m}{\phi R_n}\left(\frac{\gamma_{\text{LRFD}_i}}{Q_{m_i}}\right)\right] \tag{13}$$

where γ_{LRFD_i} is the effective LRFD load factor for the load combination under consideration, Q_{m_i} is the mean load effect multiplier for that load combination, and V_{Q_i} is the coefficient of variation of the load effect, all as a function of the varying load ratio as indicated by the subscript *i*.

To convert Equation 13 for use with ASD load combinations, γ_{LRFD_i} is replaced by γ_{ASD_i} and ϕR_n is replaced by R_n/Ω. Thus, Equation 13 becomes

$$\beta = \frac{1}{\sqrt{V_R^2 + V_{Q_i}^2}} \ln\left[\frac{R_m}{\left(R_n/\Omega\right)}\left(\frac{\gamma_{\text{ASD}_i}}{Q_{m_i}}\right)\right] \tag{14}$$

Based on extensive studies for A992 steel (Bartlett et al.[2]) and the original work for the development of the 1986 AISC *Specification* by Ravindra and Galambos, the following values can be used:

$$M_m = 1.055;\quad V_M = 0.058$$
$$F_m = 1.00;\quad V_F = 0.05$$
$$P_m = 1.02;\quad V_P = 0.06$$

Thus,

$$R_m = R_n\left(1.055\right)\left(1.00\right)\left(1.02\right) = 1.076R_n$$

$$V_R = \sqrt{\left(0.058\right)^2 + \left(0.05\right)^2 + \left(0.06\right)^2} = 0.097$$

Based on Galambos et al.[3] (1982), the ratio of mean to code specified dead and live loads can be taken as, for dead load

$$D_m/D = 1.05;\quad V_D = 0.10$$

and for live load

$$L_m/L = 1.00;\quad V_L = 0.25$$

The values for Q_{m_i} and V_{m_i} will be functions of the live-to-dead load ratio.

The reliability index, β, based on Equations 13 and 14, for a live-to-dead load ratio from 1.0 to 5.7 is presented in Figure 1.18 for a compact wide-flange beam under uniform moment for both LRFD and ASD. The figure is based on the load combination of live load plus dead load and statistical variations consistent with those used in the development of the *Specification*. It is seen that the reliability of design by LRFD is somewhat more uniform for this condition than design by ASD and that at a live to dead load ratio of approximately 3, the two approaches yield the same reliability. The higher the reliability index is, the safer the structure. Regardless of the numerical value of β, any structure that meets the requirements of the *Specification* will be sufficiently safe. A more detailed discussion of the statistical basis of steel design is available in *Load and Resistance Factor Design of Steel Structures*.[4] Since the introduction of the 2005 AISC *Specification*, design by ASD and LRFD have essentially been equivalent and differ only by the effect of load combinations.

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[2] Bartlett, R.M., Dexter, R.J., Graeser, M.D., Jelinek, J.J., Schmidt, B.J. and Galambos, T.V. (2003), "Updating Standard Shape Material Properties Database for Design and Reliability," *Engineering Journal,* American Institute of Steel Construction, Vol. 40, No. 1, pp. 2–14.

[3] Galambos, T.V., Ellingwood, B., MacGregor, J.G. and Cornell, C.A. (1982), "Probability-Based Load Criteria: Assessment of Current Design Practice," *Journal of the Structural Division*, American Society of Civil Engineers, Vol. 108, No. ST5, May, pp. 959–977.

[4] Geschwindner, L. F., Disque, R. O., and Bjorhovde, R. *Load and Resistance Factor Design of Steel Structures.* Englewood Cliffs, NJ: Prentice Hall, 1994.

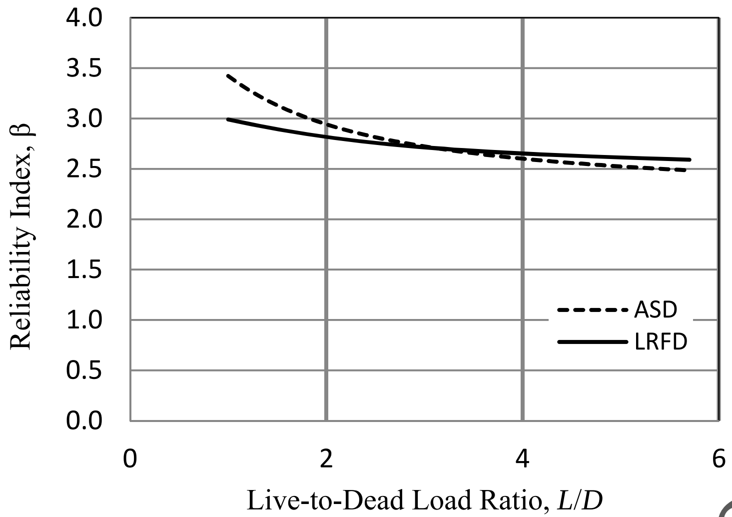

Figure 1.18 Reliability Index vs Live-to-Dead Load Ratio for Compact Simply Supported Wide-Flange Beams with Uniform Moment

General structural integrity requires a continuous load path to the ground for resisting all gravity and lateral loads that might be applied to the structure. With the introduction of the 2016 AISC *Specification*, provisions that address structural integrity beyond these general requirements have been introduced. The requirements in Section B3.9 are beyond normal strength requirements and are intended to improve the connectivity of the structure and thus the performance of the structure under undefined extraordinary events. These requirements apply only to a small set of structures where additional structural integrity is mandated.

1.13 LIMIT STATES

Regardless of the design approach, ASD or LRFD, or the period in history of the design's execution, 1923 or 2018, all design is based on the ability of a structure or its elements to resist load. This ability is directly related to how an element carries that load and how it might be expected to fail, which is referred to as the element's limit state. Each structural element can have multiple limit states, and the designer is required to determine which of these limit states will actually limit the structure's strength.

There are two types of limit states to be considered: strength limit states and serviceability limit states. Strength limit states are those limiting conditions that, if exceeded, will lead to collapse of the structure or a portion of the structure, or to such serious deformations that the structure can no longer be expected to resist the applied load. Strength limit states are identified by the *Specification*, and guidance is provided for determination of the nominal strength, R_n, the safety factor, Ω, and the resistance factor, ϕ. Examples of the more common strength limit states found in the *Specification* are yielding, rupture, and buckling.

Serviceability limit states are not as well defined as strength limit states. If a serviceability limit state is exceeded, it usually means that the structure has reached some performance level that someone would find objectionable. The *Specification* addresses design for serviceability in Chapter L and defines serviceability in Section L1 as "a state in which the function of a building, its appearance, maintainability, durability, and the comfort of its occupants are preserved under typical usage." Chapter L lists deflections, drift, vibration, wind-induced motion, thermal expansion and contraction, and connection slip as items to be considered, although no specific limitations are given for any of these limit states.

Strength and serviceability limit states will be addressed throughout this book as appropriate for the elements or systems being considered.

1.14 BUILDING CODES AND DESIGN SPECIFICATIONS

The design of building structures is regulated by a number of official, legal documents that are known commonly as *building codes*. These cover all aspects of the design, construction, and operation of buildings and are not limited to just the structural design aspects.

The model code currently in use in the United States is the ICC International Building Code. Model codes are published by private organizations and have been adopted, in whole or in part, by state and local governments as the legal requirements for buildings within their area of jurisdiction. In addition to the model codes, cities and other governmental entities have written their own local building codes. Unfortunately, since the adoption of a building code is in great part a political activity, the regulations in use across the country are not uniform. A new International Building Code is published every 3 years but not adopted as quickly as issued. Thus, building codes with effective dates from 2003 to 2015 are still in use. In addition, governmental bodies will often adopt a model code with local amendments. Because of the technical nature of the AISC *Specification*, local amendments normally do not affect those aspects of steel design but they often do modify the loading definitions and thus do ultimately affect steel design.

To the structural engineer, the most important sections of a building code deal with the loads that must be used in the design, and the requirements pertaining to the use of specific structural materials. The load magnitudes are normally taken from *Minimum Design Loads for Buildings and Other Structures*, a national standard published by the American Society of Civil Engineers (Structural Engineering Institute) as ASCE/SEI 7. The loads presented in ASCE/SEI 7 may be altered by the model code authority or the local building authority upon adoption, although this practice adds complexity for designers who may be called upon to design structures in numerous locations under different political entities. Throughout this book, ASCE/SEI 7 will be referred to simply as ASCE 7 as it is most commonly referred to in the profession.

The AISC *Specification* is incorporated into the model building code by reference. The *Specification*, therefore, becomes part of the code, and thus part of the legal requirements of any locality where the model code is adopted. Locally written building codes also exist and the AISC *Specification* is normally adopted within those codes by reference also. Through these adoptions the AISC *Specification* becomes the legally binding standard by which all structural steel buildings must be designed. However, regardless of the *Specification* rules, it is always the responsibility of the engineer to ensure that their structure can carry the intended loads safely, without endangering the occupants.

1.15 INTEGRATED DESIGN PROJECT

This section introduces a building to be used in subsequent chapters of this book as an integrated design project. It is a relatively open-ended design project in that only a limited set of design parameters are set at this point. Several options will be presented in subsequent chapters so that the project can be tailored at the desire of the instructor.

The building is a four-story office building with one story below grade. It is located in Downers Grove, Illinois, at approximately 42°N latitude and 88°W longitude. This is a 102,000 ft^2 building with approximately 25,500 ft^2 per above-grade floor. For the first three floors, the floor-to-floor height is 13 ft 6 in. For the top floor, the floor-to-roof height is 14 ft 6 in. The below-grade floor-to-floor height is 15 ft 6 in. The façade is a lightweight metal curtain wall that extends 2.0 ft above the roof surface, and there is a 6.0 ft screen wall around the middle bay at the roof to conceal mechanical equipment and roof access. All steel will receive spray-applied fireproofing as necessary.

Based on preliminary discussions with the architectural design team, the design will start with bay sizes of 30.0 ft in the east-west direction and 45.0 ft, 30.0 ft, and 45.0 ft in the north-south direction, as shown in Figure 1.19. Representative floor and roof framing plans are shown in Figures 1.20 through 1.22. To accommodate a two-story atrium on the first floor, the second floor framing plan shows an opening bounded by column lines A, C, 4, and 5. The lateral load resisting system consists of a pair of three-bay moment frames in the east-west direction and a pair of one-bay chevron braced frames in the north-south direction.

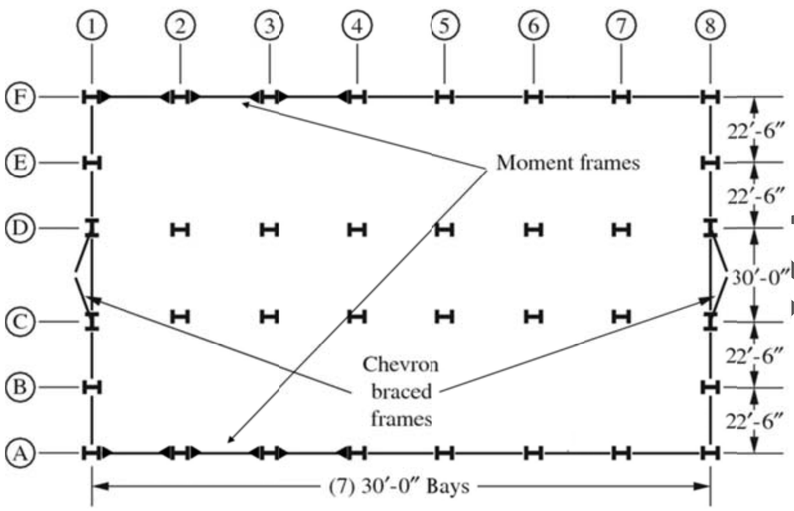

Figure 1.19 Schematic Plan for Integrated Design Problem

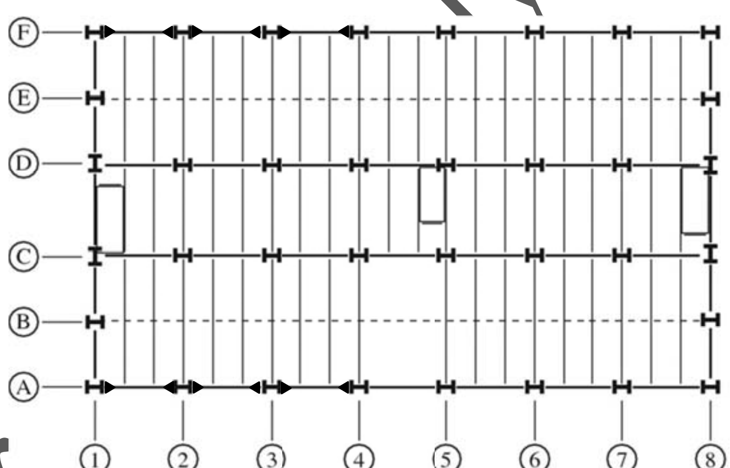

Figure 1.20 Representative Second-Floor Framing Plan

Much of today's structural analysis and design is accomplished through the use of integrated analysis and design software. Figure 1.23 shows an example of a complete three-dimensional model of the given preliminary framing system developed using RAM Structural System. Figure 1.24 shows the results of the same computer model with the gravity-only structural elements removed.

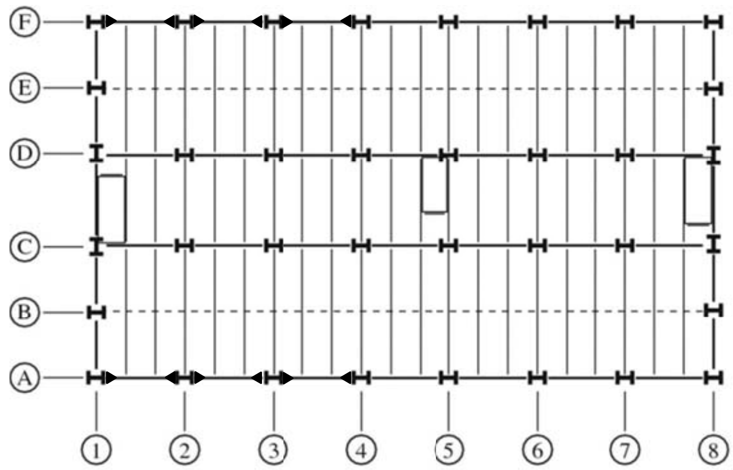

Figure 1.21 Representative First-, Third-, and Fourth-Floor Framing Plan

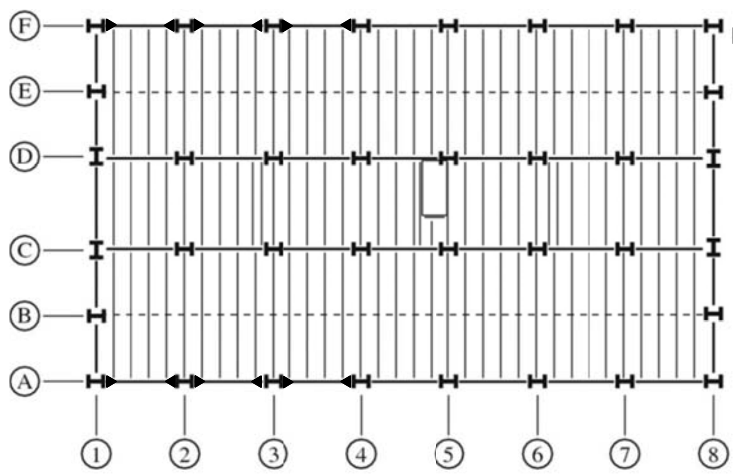

Figure 1.22 Representative Roof Framing Plan

Figure 1.23 Three-Dimensional Computer Model of Complete Structure from RAM Structural System

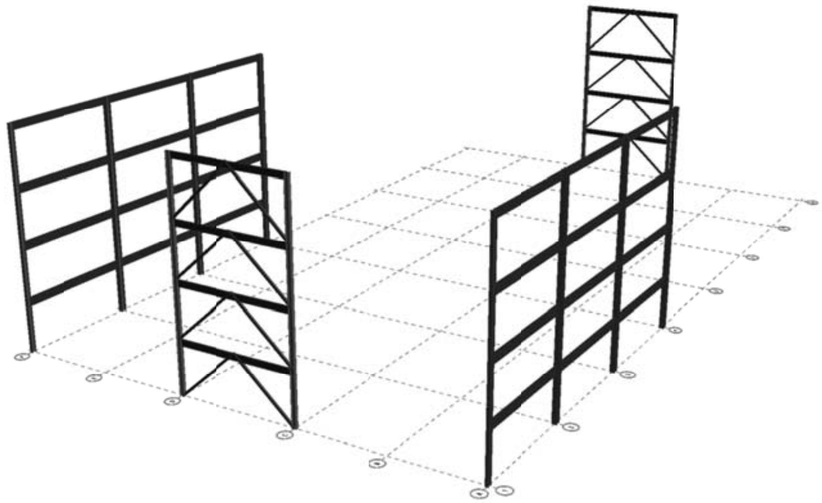

Figure 1.24 Three-Dimensional Computer Model Showing Only Lateral Load Resisting Systems from RAM Structural System

1.16 PROBLEMS

1. Where could one find information about the provisions of the 1961 AISC *Specification*?

2. What resource would be most likely to assist in the determination of properties of a steel member found in a building built in 1954?

3. Which chapter of the AISC *Specification* provides information about:

- **a.** general requirements for analysis and design
- **b.** design of members for flexure
- **c.** design of connections
- **d.** design of members for combined forces and tension
- **e.** requirements for design of structures to ensure stability

4. In the *AISC Steel Construction Manual*, where can one find:

- **a.** the AISC *Specification*
- **b.** design considerations for bolts
- **c.** dimensions and properties for structural steel shapes
- **d.** design of compression members

5. List and define the three basic goals of a design team for the design of any building.

6. All structures are composed of some or all of five basic structural types. List these five basic structural components and provide an example of each.

7. Provide an example of each of the following types of construction. To the extent possible, identify specific buildings in your own locale.

- **a.** Bearing wall
- **b.** Beam-and-column
- **c.** Long-span
- **d.** High-rise
- **e.** Gable-frame

8. What type of structural system uses the combined properties of two or more different types of materials to resist the applied loads?

9. List and describe two types of lateral bracing systems commonly used in beam-and-column construction.

10. In designing a steel structure, what must be the primary concern of the design engineer?

11. Provide a simple definition of structural design.

12. Describe the difference between a strength limit state of a structure and a serviceability limit state.

13. Give a description of both the LRFD and ASD design approaches. What is the fundamental difference between the methods?

14. Provide a brief description of plastic design (PD).

15. Identify three sources of variation in the strength of a structure and its components.

16. Provide three examples of strength limit states.

17. Provide three examples of serviceability limit states.