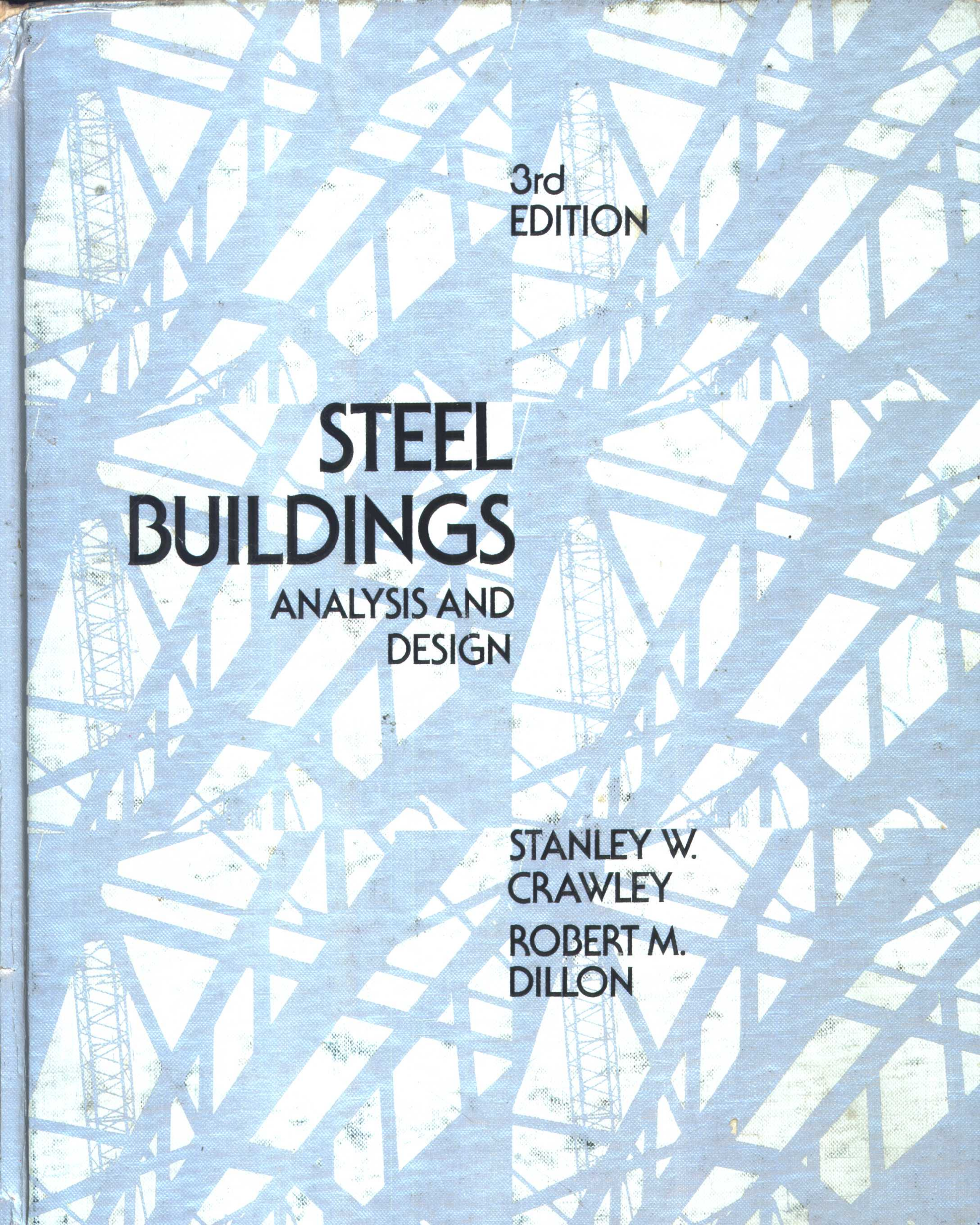


3rd
EDITION

STEEL BUILDINGS

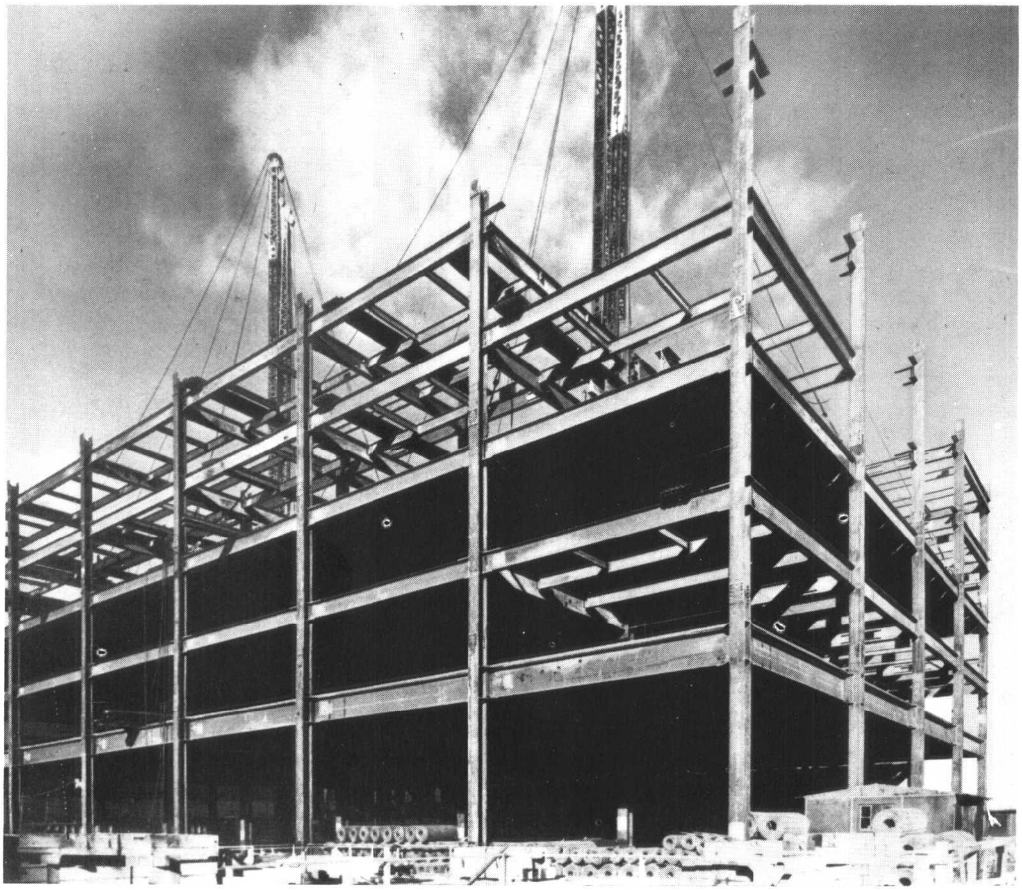
ANALYSIS AND DESIGN

STANLEY W. CRAWLEY

ROBERT M. DILLON

STEEL BUILDINGS

ANALYSIS AND DESIGN



The Borden Building, Columbus, Ohio

John Wiley & Sons

New York Chichester Brisbane Toronto Singapore

Frontispiece credits: general contractor, Turner Construction Company; architect, Harrison & Abramovitz; owner, 180 East Broad Partnership (composed of John W. Galbreath & Company and The Equitable Life Assurance Society of the United States).

Cover typography by Frank Emmi
Cover design by Kevin J. Murphy

Library of Congress Cataloging in Publication Data:

Crawley, Stanley
Steel buildings.

Includes indexes.
1. Building, Iron and steel. I. Dillon, Robert M. (Robert Morton), 1923– II. Carter, Winfred O. III. Title.

TA684.C78 1984 693.71 83-6549
ISBN 0-471-86414-5

Printed in the United States of America

10 9 8 7 6 5 4 3 2 1

Preface

The purpose of this book is to present the general principles of structural analysis and their application to the design of the more common types of low and intermediate height building frames. This third edition retains the general scope and method of presentation of the first two; however, the material has been completely updated.

A knowledge of the elementary principles of statics and strength of materials continues to be assumed, although much of this material is reviewed throughout the book as a prelude to design. Again, no attempt has been made to include complete tables of the properties of structural shapes or to give the text of pertinent codes, standards, and specifications. Sufficient information and data of this type are included, however, to enable the reader to follow the presentation without difficulty. This is not the case with many of the problems. Apart from the impracticality of supplying additional information and data of this kind, it is important that the reader become familiar as early as possible with the various reference materials used in practice.

The Manual of Steel Construction (American Institute of Steel Construction) is recommended for use in conjunction with this book. Most of the discussion and illustrative examples, as well as answers to problems, have been keyed to the 1980, eighth edition of the *Manual* and to the November 1, 1978 AISC Specification, which it contains. Reference also is made to the major model building codes, to ANSI and ASTM (American National Standards Institute and American Society for Testing and Materials, respectively) to the Steel Joist Institute, and to AISI (American Iron and Steel Institute), where their recommended requirements or procedures seemed appropriate. Even though every effort has been made to include the most recent data, it can be expected that code, standards, and specification changes will continue to be made as these organizations endeavor to keep abreast of newly acquired knowledge and experience. These newer references should, of course, be sought out, evaluated, and used. It is our intention to update this textbook again when new information becomes available to an extent that major change is necessary, and when further or more rigorous analytical and design techniques are in need of assimilation.

The book's material and method of presentation have remained basically unchanged. As in the first and second editions, our intent is to bridge the gap between academic work and professional practice—that is, to carry analysis and design beyond that applied to individual members and components and into total structural frames.

Dr. Winfred O. Carter's name is joined with ours on the title page for his considerable contributions in the development of Chapter 12 on computer-aided design and supporting appendixes in the second edition, and for his review of those materials for this edition. The two programs given in Appendixes D and E have broad application and can be transferred to punched cards for ready use; or, if preferred, the punched cards can be obtained directly by contacting the authors. We again express our appreciation to Dr. Carter and to the users of the second edition who have contributed helpful suggestions. We thank those who, at the request of John Wiley & Sons, reviewed the second edition for the purpose of suggesting improvements in this third edition—Professors Carrol D. Claycamp, Texas A & M University, Anthony J. Dasta, University of Florida, John M. McCormick, Columbia University, Charles M. Milne, Montana State University, and Edward P. Reidy, Wentworth Institute of Technology. We also thank those who reviewed the manuscript of this third edition—Professor

Anthony J. Dasta again; Professors Matthew W. Fuchs, Milwaukee School of Engineering, Achintya Haldar, Georgia Institute of Technology, John H. McMillan, Mohawk Valley Community College, and William P. Ross, Kent State University; and Charles N. Timbie, Structural Engineer.

Appreciation is expressed to the American Institute of Steel Construction, the American Iron and Steel Institute, the Steel Joist Institute, the American National Standards Institute, and the International Conference of Building Officials for their continued cooperation in making inclusion of reference materials possible.

STANLEY W. CRAWLEY
ROBERT M. DILLON

Washington, D.C.

STEEL BUILDINGS

ANALYSIS AND DESIGN

Contents

12
Computer-Aided Analysis and Design 443

13
Building Design Project 509

Appendixes

Index 663

1

GENERAL CONSIDERATIONS

1.1 Introduction

Modern buildings are constructed in many different ways. However, when speaking of buildings in a structural sense, the majority can be classified as wall-bearing, skeleton-frame, or a combination of both. With wall-bearing construction, floors and roof are supported by load-bearing walls; the thicknesses of the walls are determined largely by the number of stories and the magnitude of the loads thus brought to them for support. With skeleton-frame construction, walls as well as floors and roof are supported by a structural framework of beams, girders, columns, and similar members.

Until the past few years, wall-bearing construction generally had ceased to be economical and practical in the United States except for low-rise buildings. Indeed, it was the practical and economical limitations of such construction when applied to medium-rise and high-rise buildings that led to the development of the skeleton-frame. Today, however, with the emergence of higher strength masonry units and, particularly, high-bond mortars, there has been a resurgence of interest in wall-bearing, high-rise construction. There also is considerable interest in a variety of other construction concepts that do not rely, or rely only in part, on a skeleton-frame for support—e.g., various load-bearing, panelized, and volumetric module solutions for low-rise buildings and even for buildings of considerable height. In addition, it can be anticipated that much greater attention will be given in the years ahead to construction concepts based on dimensionally and functionally coordinated "subsystems"—i.e., exterior wall, floor-ceiling, partitioning, and similar building elements that can be assembled to create buildings of a wide variety of sizes and types. Structural integration will become extremely important, and the skeleton-frame can be expected to play an important role.

The skeleton-frame, therefore, will probably continue to be as significant in the future as it is in today's construction. Because the exterior walls in skeleton-frame construction are relieved of a load-carrying function, they serve primarily as an enclosing environmental control envelope—i.e., to control light, temperature, sound, moisture, etc. Freeing the exterior walls of a primary structural function permits a much wider choice of materials and methods of fabrication. The skeleton-frame also can free interior walls and partitions of a primary structural role, and doing so allows much greater flexibility in architectural planning, including the ability to plan for changeable interior space. The skeleton-frame also offers advantages in terms of ease of fabrication and transport, speed of erection, and coordination of building trades. All of these characteristics are invaluable in meeting today's complex building requirements.

Although the structural frame for any given building will be developed from the standpoints of structural adequacy and economy, column locations and spacing will usually be determined by architectural considerations arising from anticipated as well as immediate occupancy requirements. It is important to recognize that, just as there is generally more than one architectural scheme that will satisfy occupancy requirements, so also will there be several satisfactory structural solutions. The architectural and structural schemes should be compatible, i.e., they should "build" well without resorting to unduly complex and extravagant structural arrangements. (See also Art. 13.1.)

In multistory buildings of skeleton-frame construction, the most economical center-to-center column spacing for average loads is on the order of 22 to 28 ft. Spacings less than 20 ft are seldom economical from a structural standpoint, and those 30 ft and more, even in one direction, can usually be justified only when occupancy considerations call for the greater unobstructed floor area that such spans provide. These guidelines do not necessarily apply to high-rise (tall) buildings—such buildings frequently pose unique structural problems.

Although the principles developed in this text are generally applicable to all types of steel frame building, as mentioned in the Preface, structural design emphasis is placed upon the more common types such as commercial and industrial buildings, apartment houses, schools, hospitals, and similar structures.

1.2 Design Procedure

To reiterate, it is of utmost importance that the structural design of a building be coordinated with both the architectural scheme and the mechanical-electrical requirements from the inception of the project. The general arrangement of floor framing, and especially the placement of columns, should be borne in mind during development of the architectural scheme. Preliminary framing plans should be made and column dimensions approximated before a final scheme is adopted. This is necessary because the size of columns and the clearances required, especially in the lower stories, may materially affect the architectural layout. As soon as the floor framing arrangement has been determined, beams and girders can be designed, followed by the final design of columns and foundations. Throughout development of the structural design, the architectural, structural, and mechanical-electrical plans must constantly be checked against one another to insure accuracy and overall efficiency of design and construction, and ultimate building operation.

Although for small buildings the structural framing is sometimes shown directly on the architectural plans, this practice is not recommended. For projects of any appreciable size, a separate set of framing plans is essential if the location, size, and joining of structural members are to be recorded and made readable without the confusion of other information. On all but the smallest projects, the modest amount of time and effort required to provide separate framing plans is well worth the effort. The general character of framing plans and their relationship to the architectural drawings will be readily apparent from a brief study of the set of working drawings for the building presented in Chapter 13.

1.3 Design Loads

The loads for which a building is designed are classified as dead, live, and environmental. In the past, environmental loads were frequently considered part of the live load—e.g., those due to wind, snow, rain, earthquake forces, and soil and hydrostatic pressures (the latter two acting horizontally on walls below grade). However, current practice tends to recognize these as three distinct classifications.

Dead loads / These are the loads due to the weight of the permanent parts of the building such as floors, beams, girders, walls, roof, columns, stairways, fixed partitions, etc., and include fixed service equipment such as mechanical and electrical system components, water tanks, and similar items supported by the structure. The weights of different building materials to be used in determining the dead loads are specified in local building codes. Where such data are incomplete, or in localities where no code is operative, the reader is referred to the comprehensive lists given in the model building codes or the American National Standards Institute's *Minimum Design Loads for Buildings and Other Structures*, ANSI A58.1–1982.[1] For example, a list of cubic foot weights of representative basic building materials, most of which are as recommended in that publication, is given in Table 1.1. The net effect of any prestressing of members also is classified as a part of the dead load.

[1] Available from ANSI at 1430 Broadway, New York, N.Y. 10018.

Live Loads / These represent the probable loads on the building structure due to occupancy and use, and generally are considered to be uniformly distributed over the floor area. The value assessed, expressed in pounds per square foot (psf), is enough to cover the effect of ordinary concentrations that may occur. Except for the dead load, all vertical loads are included, e.g., the weight of occupants, furniture, other than fixed equipment, and stored materials. On roof surfaces, it includes an allowance for maintenance workers and equipment, and movable objects and people. Buildings that will contain heavy machinery or similar large concentrations of live load must, of course, be designed specifically for such concentrations.

There continues to be a lack of uniformity among different building codes as to proper live load allowances for various types of occupancy; however, there are increasing efforts by a number of organizations to bring about an ever-higher degree of uniformity.[2] Presented in Table 1.2 are typical recommended live load allowances for various occupancies abstracted from the Chicago building code and several well-known model codes and standards. A re-

[2] Some of these are: the Conference of American Building Officials (CABO), including the International Conference of Building Officials (ICBO), Building Officials and Code Administrators, International (BOCA), and Southern Building Code Congress, International (SBCC); and the American National Standards Institute (ANSI). In addition, the Congress of the United States, as part of the Housing and Community Development Act of 1974, authorized creation of the now functioning, private, nongovernmental National Institute of Building Sciences (NIBS). One of NIBS's primary objectives is to achieve appropriate uniformity in building codes and standards. Currently, ICBO, headquartered in California, has achieved state and local governmental adoptions of its code principally in the Far West; SBCC, principally in the South and Southwest; and BOCA, principally in the North Central and Northeast. However, there is a considerable overlap.

Table 1.1
Weights of Typical Building Materials

Materials	*Pounds per Cubic Foot*	*Materials*	*Pounds per Cubic Foot*
Cast-stone masonry (cement, stone, sand)	144	Masonry, rubble mortar:	
Cinder fill	57	Granite	153
Concrete, plain:		Limestone, crystalline	147
Cinder	108	Limestone, oolitic	138
Expanded-slag aggregate	100	Marble	156
Haydite (burned-clay aggregate)	90	Sandstone	137
Slag	132	Metals	
Stone (including gravel)	144	Aluminum	165
Vermiculite and perlite aggregate, non-load-bearing	25–50	Bronze	710
Other light aggregate, load-bearing	70–105	Lead	710
Concrete, reinforced:		Steel	490
Cinder	111	Terra cotta, architectural:	
Slag	138	Voids filled	120
Stone (including gravel)	150	Voids unfilled	72
Earth	80–120	Timber, seasoned:	
Masonry, ashlar:		Ash, commercial white	41
Granite	165	Cypress, southern	32
Limestone, crystalline	165	Fir, Douglas, Coast region	34
Limestone, oolitic	135	Oak, commercial reds and whites	45
Marble	173	Pine, southern yellow	39
Sandstone	144	Redwood	28
Masonry, brick:		Spruce, red, white, and Sitka	28
Hard (low absorption)	130		
Medium (medium absorption)	115		
Soft (high absorption)	100		

duction in these live loads for large areas generally is permitted.

Some codes also require that provision be made in the live loads of office and loft buildings for the effect of partitions, which may be either movable or not fixed as to location until after the building is erected. When this is the case, an allowance of 20 psf of floor area is often used. In addition, codes usually stipulate minimum concentrated live loads.

Special provision also must be made for impact loads such as those caused by the operation of elevators. It is common practice, for example, to provide for this impact effect in the design of beams, girders, and the first tier of columns supporting elevator machinery, by increasing the actual loads a specified percentage—e.g., 100 per cent. Also, it usually is required that accessible roof-supporting members be designed to support a concentrated load of some 2000 lb that may be suspended from them.

Environmental Loads / Included under this rather recent load classification are those previously mentioned—i.e., wind, snow, rain, earthquake forces, soil and hydrostatic pres-

Table 1.2
Typical Minimum Live Load Requirements

Classes of Occupancy	Minimum Live Loads per Square Foot (psf) of Floor Area: City of Chicago 1977	Natl. Bldg. Code 1967[a]	Std. Bldg. Code 1976[b]	Basic Bldg. Code 1979[c]	Uniform Bldg. Code 1982[d]	Amer. Natl. Std. Min. Design Loads 1982[e]
Assembly						
Fixed seats	60	60	50	60	50	60
Movable seats	100	100	100	100	100	100
Dwellings	40		40[f]		40	
First floor	—	40	—	40	—	40[i]
Habitable second floor	—	30	—	30	—	30[i]
Garages						
Passenger car	50–100	50	120[g]	50	50[g]	50
Other	—	150–200	—	—[g]	100[g]	—[g]
Hospitals						
Private rooms	40	40	40	40	40	40
Operating rooms	40	60	—[h]	60	—[g]	60
X-ray rooms	—	100	—[h]	—	—[g]	—
Hotels						
Guest rooms	40	40	40	40	40	40
Manufacturing	100					
Light	—	125	100	125	75[g]	125
Heavy (factories)	—	125	150	250	125[g]	250
Office space						
Typical rooms	50	80	50	50	50	50
Schools						
Class rooms	40	40	40	40	40	40
Sidewalks						
Over areaways, etc.	—	—[g]	200[g]	250	250[g]	250
Stores	100					
Retail	—	100	75	75–100	75[g]	75–100
Wholesale	—	125	100	125	100[g]	125
Warehouses	100	125–250	125–250	—	125–250	125–250

[a] American Insurance Association (*Note*: The rights to this code were acquired by the National Conference of States on Building Codes and Standards in 1980 and then by BOCA in 1982).

[b] Southern Building Code Congress, International (SBCC).

[c] Building Officials and Code Administrators, International (BOCA).

[d] International Conference of Building Officials (ICBO).

[e] American National Standards Institute (ANSI); *Minimum Design Loads for Buildings and Other Structures*, A58.1—1982.

[f] Except sleeping rooms and attics with storage, where the specified load is 30 psf.

[g] Requirements for maximum wheel loads, and/or concentrated or special loads.

[h] To be approved by the building official.

[i] Habitable attics and sleeping areas, 30 psf; all other areas, 40 psf.

Note: In most codes there are additional classes and/or subclasses of occupancy; also, in many cases, there are additional qualifiers that could be shown. The actual documents should be referred to.

sures, and self-restraining forces such as those caused by temperature and moisture.

Building codes generally specify minimum wind pressures which a building must be designed to withstand. The design of buildings for wind is introduced in Chapter 8.

The snow load is an essential element of the load on roofs in many geographical areas. Maps indicating ground snow loads in pounds-force per square foot for various parts of the United States are shown in ANSI A58.1—1982. Snow loads and their computation are discussed in greater detail in Chapter 9. Rain loads, ponding of water on roofs, and the effects of rain on snow are also treated in Chapter 9.

In many localities, earthquake resistance is a critical factor, and building codes in these areas generally require earthquake-resistant design and/or special details. Such attention to earthquake effects is well known in the western United States. Loads resulting from earthquakes are discussed in some detail in Chapter 8.[3]

1.4 Working Stresses

Building codes are in much better agreement regarding allowable values for structural steel working stresses. The term *working stress* is a carry-over from early design procedures and can be somewhat loosely defined as the unit stress to which the steel will be subjected in actual use (based on elastic performance of the structure). The *allowable* working stress is considerably less than the breaking strength of the steel. The better agreement in codes largely results from continued activity over many years by professional and technical societies and trade associations. The allowable working stresses recommended by the American Institute of Steel Construction (AISC) are now widely accepted throughout the United States. These stress levels are listed in the 1978 AISC "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings."[4]

In the 1978 AISC Specification, the allowable working stress is listed as a percentage[5] of the *yield* stress which varies with each type and grade of steel. Since the yield stress is closely associated with that point where permanent deformation takes place, it is a basic and important physical property of the material and a guide to its design strength (Appendix B). Furthermore, the allowable working stress depends on the type of stress under consideration, i.e., axial tension, bending, shear, etc. For example, except in the case of plates over 8 in. in thickness, a steel bearing the designation A36 (Art. 1.6) has a yield point of 36,000 lb per sq in. (psi) and, under certain circumstances, would have an allowable working stress in bending that is 60 per cent of 36,000 or 21,600 psi, which is rounded off to 22,000 psi. It is of interest to note that the tensile breaking strength of this same steel is between 58,000 and 80,000 psi.

Although the design of light-gage steel members is not specifically discussed in this text (except for joists covered in Chapter 5), the attention of the reader is directed to the following applicable specifications:

Standard Specifications for Open Web Steel Joists, issued by the Steel Joist Institute.

Specification for the Design of Light Gage Cold-

[3] A new organization, the Building Seismic Safety Council (BSSC), was created in 1979 under the auspices of the National Institute of Building Sciences (NIBS) to bring together the many U.S. building community organizations to foster the development and appropriate applications of improved seismic safety provisions in building design and regulations throughout the United States. Evidence of this work and that of the National Bureau of Standards, which is involved in the BSSC work and provided the Secretariat for ANSI A58.1, is already apparent in the content of the 1982 A58.1 standard.

[4] Contained in the *Manual of Steel Construction*, American Institute of Steel Construction, Eighth Edition, and available separately from AISC, 400 North Michigan Avenue, Chicago, Illinois 60611.

[5] Under certain circumstances, the working stress is listed as a function of the modulus of elasticity.

Formed Steel Structural Members and *Specification for the Design of Light Gage Cold-Formed Stainless Steel Structural Members*, both issued by the American Iron and Steel Institute.

The 1978 AISC Specification is still quite recent; therefore, the designer may still encounter local building code jurisdictions that do not permit its stress values. The local building code should always be consulted in actual design work.

1.5 Factor of Safety

No structural member of a building frame is ever designed to carry a load that will develop its full ultimate strength under normal service conditions. There are too many elements of uncertainty, both as to loading and uniformity in quality of materials and construction to permit such a degree of precision in structural design. Consequently, some margin of safety must be provided, and this is accomplished by setting allowable working stresses at values well below the ultimate strength. The ratio between ultimate strength and working stress has been defined as the *factor of safety*. However, this definition is not wholly satisfactory, since failure of a structural member in a building actually begins when the stress exceeds the yield point (or more precisely the elastic limit). This is due to the fact that deformations produced by stresses above this value are *permanent* and thus change the shape of the structure, even though there may be no danger of collapse. Thus, even though there is no general agreement on an exact definition of *factor of safety*, the above discussion will serve to indicate the concept. The relationships among ultimate strength, elastic limit, yield point, and deformation under stress are reviewed briefly in Appendix A and in Chapter 11, Ultimate Strength and Plastic Design.

1.6 Structural Steels

There is a wide variety of structural steels available to designers today. This was not the case 15 years ago. Major improvements have been made in strength, ductility, and corrosive resistance. As noted in Art. 1.4, the yield stress becomes the index for structural strength, and for many years, the yield stress for structural steel was effectively limited to 33,000 psi. Today, structural steel is readily available in a range of yield stresses from 32,000 to 130,000 psi and, in the near future, are expected to be available in yield stresses reaching 160,000 to 200,000 psi.

Ductility is the ability of a material to flow with a constant (or nearly constant) stress and still maintain its strength. The greater the ductility, the more a structure can adjust to peak stress resultants, thus providing more reserve strength. This characteristic forms the basis for plastic design, which is presented in Chapter 11. It should be pointed out, however, that except for certain earthquake specifications, at present there is no required minimum ductility for steel structures.

Corrosion-resistant steels and new techniques for fireproofing steel members have significantly altered the appearance of steel buildings in recent years. Steels that are not corrosion resistant cannot be left exposed to the weather without painting and maintenance; not only will they rust and look unsightly but also the steel will lose strength as a result of corrosive action. However, when the new corrosion-resistant or "weathering" steels are used, a natural oxide coating quickly forms on exposed surfaces, protecting the steel much the same as painting. Care must be taken with architectural detailing, however, to be sure that moisture on such steel surfaces will be properly drained away so as not to streak windows or stain concrete and other surfaces.

In addition, it must always be remembered that steel will begin to lose its load-carrying ability when its temperature reaches 600°F and will